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	Engineering and Design DESIGN OF SMALL WATER SYSTEMS	
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DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

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Manual
No. 1110-2-503

27 February 1999

Engineering and Design
DESIGN OF SMALL WATER SYSTEMS

1. Purpose. This manual provides guidance and criteria for the design of small water supply, treatment, and distribution systems. For the purpose of this manual, small water systems shall be those having average daily design flow rates of 380,000 liters per day (100 000 gallons per day) or less.

2. Applicability. This manual is applicable to all USACE commands concerned with water source development and the design of water treatment and distribution systems.

3. General. This manual provides information of interest to planners and designers of small water systems. Such systems generally cannot benefit from economies of scale, and proper management and operation are critical to produce satisfactory finished water quality. The major emphasis of this manual is on the design of systems that will be effective and reliable, requiring a level of operation and management activity commensurate with their physical size and the available sources.

4. Distribution. Approved for public release, distribution is unlimited.

FOR THE COMMANDER:

ALBERT J. GENETTI, JR.
Major General, USA
Chief of Staff

This manual supersedes EM 1110-2-503, dated 25 January 1984.

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Chapter 1
Introduction

1-1. Purpose

This manual provides guidance and criteria for the design of small water supply, treatment, and distribution systems. For the purpose of this manual, small water systems shall be those having average daily design flow rates of 380 000 liters per day (l/d) (100 000 gallons per day (gpd)) or less. However, the use of the term small is arbitrary, there being no consensus in the water supply literature with respect to its meaning. Regulations regarding the acceptability of a water source, degree of treatment required, and the monitoring requirements are not based on flow rates, but rather on a water system classification relating to the number of people served and for what period of time. Figure 1-1 provides a flowchart for system classification. Refer to Chapter 3, paragraph 3-4*b* for the appropriate nomenclature.

1-2. Applicability

The provisions of this manual are applicable to USACE commands concerned with water source development and the design of water treatment and distribution systems for civil works projects. The provisions of Army Regulation 200-1, Environmental Quality: Environmental Protection and Enhancement, shall be adhered to during the design of any civil works activity under the jurisdiction of the U.S. Army Corps of Engineers.

1-3. References

Required and related publications are listed in Appendix A.

1-4. General Considerations

a. Background. Historically the U.S. Army Corps of Engineers has been concerned with providing potable water to the public at its various recreation facilities. The passage of the Safe Drinking Water Act (SDWA) of 1974 (PL 93-523) (U.S. Congress 1974) and its subsequent amendments has placed new constraints and requirements on all sectors of the water supply industry and has resulted in a continuing critical review and reexamination of the entire potable water supply system from initial source development to final delivery at the user's tap. This process is taking place in an atmosphere charged with intense public interest in the complex relationships that apparently exist between environmental quality and public health and against the backdrop of actual or potential water shortages in many locations. The reauthorized SDWA (August 1996) requires Federal agency compliance. It requires that any Federal agency comply with the SDWA in the same manner as all other drinking water systems. Under the reauthorization, sovereign immunity would be waived to allow citizens and states to seek penalties for violations at Federal facilities.

b. Emphasis. This manual provides information of interest to planners and designers of small water systems. Such systems generally cannot benefit from economies of scale, and proper management and operation are critical to produce satisfactory finished water quality. Therefore, the major emphasis of the manual is on the design of systems that will be effective and reliable, but that require a level of operation and management activity commensurate with their physical size and the available resources. To this end, consideration is given in subsequent chapters to preliminary planning, source selection and development, water quality and quantity requirements, treatment, pumping, storage, and distribution. Throughout the manual an effort is made to focus on requirements and standards, key design elements, and generalized alternative design methods and their applicability. Thus, comprehensive step-by-step design procedures are not presented. Rather, appropriate references are identified in the text and listed in Appendix A.

c. Intended use. The design of any water system depends on many factors, not the least of which is the intended use of the finished product. The information presented herein applies most directly to the design of systems to supply potable water to the public at Corps recreation facilities. However, the manual should be of interest and use to planners and designers of other small water systems such as those that may serve small communities, highway rest areas, camps, and state parks.

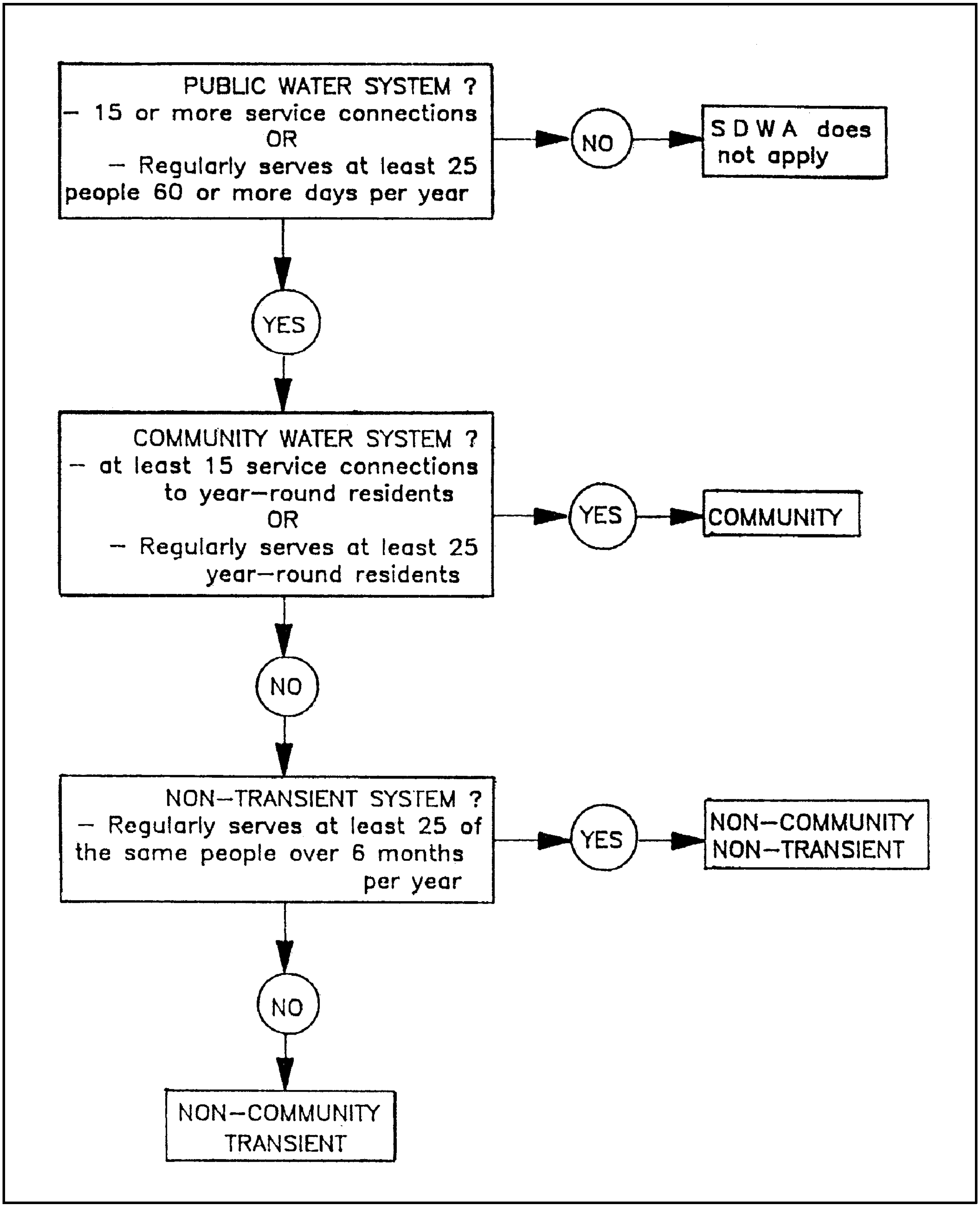

Figure 1-1. Water system categories

Chapter 2
Preliminary Planning and Design

2-1. Introduction

The function of a water supply system is to provide water from a source, treat the water to render it suitable for its intended use, and deliver the water to the user at the time and in the quantity desired. Since such factors as the yield and quality of raw water sources; topography, geology, and population density of service areas; and intended uses of water may vary, it is obvious that not all water systems will be alike. Nevertheless there are certain general considerations that designers of virtually all water systems must take into account. In this chapter the more important of these common concerns are identified and briefly discussed.

2-2. Agency Involvement

Legislation and regulation pertaining to design, construction, and operation of water systems vary considerably among the states and, in some cases, within a given state. In addition, water systems often impact in some manner on the functions and responsibility of an amazingly diverse array of legal entities. Thus the importance of early consultation and coordination with affected groups, especially state planning and public health agencies, cannot be overemphasized. A listing of such groups that might be involved in approval of at least some aspects of the design, construction, or operation of a given water system would include the following:

a. U.S. Environmental Protection Agency (USEPA).

b. State public health agency.

c. Local public health agency.

d. State pollution control agency.

e. Local pollution control agency.

f. State planning agency.

g. Local planning agency.

h. State highway department.

i. Local highway department.

j. Electric power utility company.

k. Telephone utility company.

l. Natural gas utility company.

m. Railroad company.

n. U.S. Department of Agriculture Farmers Home Administration.

o. U.S. Department of Housing and Urban Development.

p. U.S. Department of the Interior.

q. State Recreational Development Agency.

r. U.S. Army Corps of Engineers.

While the foregoing list is not all-inclusive or universally applicable, it is sufficient to make the point that obtaining all the permits and approvals necessary to actually put a water system into service is no simple matter. This is especially true since the requirements of the various groups involved will often be conflicting. These and other difficulties can usually be worked out to the satisfaction of all parties if they are addressed early on. If not, expensive and time-consuming revisions and changes in the design, construction, and/or the operation of the system will be the likely result.

2-3. Water Quality

Water quality requirements are directly related to intended use. The highest intended use considered in this manual is human consumption. Thus, it is assumed that all the water supplied must meet or exceed appropriate local, state, and Federal drinking water standards. These standards include microbiological, chemical, radiochemical, and aesthetic requirements that are applicable to water sources as well as finished waters. However, different classifications of water systems are subject to varying levels of regulation. A more detailed discussion of raw and finished water quality requirements and the legal basis for them is presented in Chapter 3. The quality of available water sources is often a very important factor in water system planning and design.

2-4. Water Quantity

A reasonably accurate estimate of the amount of water that must be supplied is needed early on in the planning stage of project development. The average daily demand is especially important since it may be used to assess the ability of available sources to meet continuing demands and to size raw water storage facilities that may be required to meet sustained demands during dry periods. Later, during the actual design process, the peak demand must be known to properly size pumps and pipelines, estimate pressure losses, and determine finished water storage requirements so that sufficient water can be supplied during peak demand periods. As a general rule, the smaller the water system, the greater the ratio of peak to average demand rates. Thus, design of small water systems is often influenced more by peak demand than average use. Methods for determining design flow rates differ for various types of water systems and are discussed in some detail in Chapter 4.

2-5. Water Sources

a. There are four alternative sources of water that are generally suitable for very small water systems:

(1) Direct connection to an existing water system.

(2) Indirect connection to an existing water system (water hauling).

(3) Development of groundwater resources.

(4) Development of surface water resources.

b. During the planning stage of project development, each potential source should be carefully evaluated in light of the water quantity and quality requirements already mentioned. The final choice of source will depend on many factors, including the following:

(1) Proximity and capacity of existing systems.

(2) Necessary institutional arrangements for obtaining water from existing systems.

(3) Yield and quality of available ground and surface water sources.

(4) Level of operation and management activity that is reasonable for the water system being designed.

c. The source of water is an important factor in deciding which environmental regulations apply. There are basically three classifications: groundwater, groundwater under the influence of surface water, and surface water. Generally, surface water and groundwater under the influence of surface water are regulated together.

d. In the vast majority of cases, operation and maintenance considerations will point toward connection to an existing system. Unfortunately, this is often infeasible or impractical because of the expense of the connecting pipeline required or institutional difficulties. In such situations, water hauling should be seriously considered, especially for small recreational areas with highly seasonal demands. Ultimately the choice usually focuses on taking water from a surface source such as a stream, lake, or reservoir, or tapping ground-water resources via wells. There are distinct advantages and disadvantages to both methods, which are discussed, along with other important considerations, in Chapter 5.

2-6. Water Treatment

The degree of treatment that a given water will require prior to routine use for human consumption depends primarily upon the initial quality of that water. Since natural water quality may vary widely between sources, and from day to day for a given source, treatment requirements also vary. In Chapter 6 commonly used water treatment processes are discussed. Emphasis is placed on simple, low-maintenance approaches that require minimal operator time and skill. Operation of complex water treatment facilities represents a major problem for the typical small water system. Thus, careful attention must be given to designing a treatment system that is compatible with the available operation and maintenance resources.

2-7. Pumping, Storage, and Distribution

Pumping, storage, and distribution facilities are needed to deliver treated water to users in response to widely varying rates of demand. Since all three components must work together to serve this purpose, their designs must be carefully integrated. Thus, material pertinent to them has been consolidated into a single chapter (Chapter 7). This chapter also contains some discussion of raw water pumping, in-plant pumping, raw water storage (also discussed in Chapter 4), and raw water transmission.

2-8. Cost Estimating

Cost estimating is a natural part of virtually any design project. Detailed cost estimates can be made only after design is fairly complete and quantities can be determined from the plans and specifications. However, it is often necessary to estimate costs early on in the planning stage of project development. Therefore, while detailed discussions of various cost estimating techniques are not presented in this manual, it is recognized that designers and planners have need for access to the most up-to-date information available. To this end, equipment manufacturers and suppliers are excellent sources of cost information.

2-9. Project File

During the course of project development, several documents must be prepared. Examples include the feasibility study, preliminary engineering report, and final engineering report (with plans, specifications, and contract documents). Generally each of these represents continuing progress in arriving at a final design. The development of each document is facilitated by the ready availability of a well-organized project file. Therefore, it is important to maintain, in a single location if at all possible, up-to-date copies of all pertinent information (reports, maps, correspondence, permits, design notes) relative to the project. This is especially useful when a lengthy period of time transpires between initial project planning and the preparation of the final design and/or when different engineers (or planners) work on different portions or phases of the project. Each engineer or planner involved with the project should place sufficient information in the file so that a person knowledgeable in the area of water system design can understand what has transpired and the current status of the project by reviewing the file contents.

2-10. Operation and Maintenance

a. Operation and maintenance, per se, are not the responsibility of project designers. However, the careful consideration of operation and maintenance is a very important aspect of design. As a general rule, small water systems should be designed to require the minimum level of operation and maintenance that is commensurate with satisfactory delivery (quantity and quality) of water to the users. This requires the designer to give ample consideration to the reliability of processes and equipment, to anticipate the types of failures that are likely to occur, and to make provisions for dealing with them with as little disruption in service as is possible. Failure to anticipate and make adequate provisions for dealing with failures is perhaps the most common shortcoming in the design of the typical small water system. System designers should always seek input from the current or future system manager/operator to learn the necessary manpower restrictions placed on the facility and how these shortfalls might be alleviated. In regard to operation and maintenance, a more complex, least capital costs system may not be superior to a less complex, more costly system. ER 200-2-3, Environmental Compliance Policies, Chapter 7, Operating Potable Water Systems at USACE Projects and Facilities, should be reviewed by the designer previous to beginning a design in order to assess the requirements placed on operation and maintenance personnel at Corps facilities.

b. Making the system more complicated than is absolutely required is probably the next most common error. In order to help the operator of the system cope with problems that may arise, it is common practice for the designer to provide him with a system operation manual and for the supplier to furnish an operation and maintenance manual for each piece of equipment. These documents should provide instructions for operating the system under various scenarios (e.g., normal, peak demand, and minimum demand periods as well as various types of emergency situations), preventative and routine maintenance procedures, and troubleshooting. If the operation and maintenance manual is developed as the project is being designed, rather than after final plans are prepared, as is often the case, many pitfalls can be avoided. In writing the manual it is important to remember that many small water systems are operated by fairly nontechnical, part-time personnel. Thus, unambiguous, clearly explicit instructions should be given.

c. Selection for optimum design technology should not be based solely on the finished water product, but also on the post-treatment residuals created during water treatment. Residuals can be in the form of sludges, backwash waters, and spent chemicals among other things. The costs for pollution control, storage, transportation, personnel training, and ultimate disposal of affected post-treatment residuals must be included in any life cycle cost analysis. After investigation, technologies that may be attractive for water treatment purposes may indeed be unacceptable due to expensive monitoring or disposal requirements for residuals. Waste minimization must be a primary factor in technology selection. Corps of Engineer facilities are assessed annually for environmental compliance. Treatment technologies that produce potentially hazardous post-treatment residuals are not favored. Designers must coordinate with facility personnel to examine the feasibility and impact, both in cost and operation and maintenance, of post-treatment residuals. The designer must determine the Federal, state, and local environmental requirements regarding hazard classification; storage; transportation; and disposal of post-treatment residuals and provide full disclosure to facility managers.

Chapter 3
Water Quality Requirements

3-1. Introduction

Water, even treated water, may be used economically for a multitude of purposes. However, water quality requirements are usually dictated by the highest level of intended use. In this manual, the highest use considered is human consumption; therefore, primary emphasis is placed on drinking water quality requirements. Water that is suitable for human consumption is of high enough quality to serve most commercial and many industrial activities. When higher quality water is required, point-of-use treatment is generally preferable to providing additional treatment of the entire water supply. Exceptions to this rule may occasionally arise, for example, when a small water system serves a relatively large commercial or industrial customer.

3-2. The National Safe Drinking Water Act

a. Purpose. The overall purpose of the SDWA is to assure that water supply systems serving the public meet certain minimum national standards. The act directs the USEPA to establish a regulatory program and to enforce it in such a way as to provide for uniform safety and quality of drinking water in the United States. Specifically the act requires USEPA to do the following:

(1) Set standards for drinking water specifying maximum permissible levels of contamination and minimum monitoring frequencies.

(2) Protect sole or principal sources of drinking water from contamination by federally assisted projects.

(3) Protect underground drinking water sources from contamination by underground injection.

(4) Establish regulatory programs for assuring compliance with the standards.

(5) Ensure proper implementation of the regulatory program through oversight and technical assistance to the states or, if necessary, through direct implementation.

(6) Provide financial assistance to the states in their implementation of programs.

(7) Gather pertinent information pertaining to drinking water sources and supplies.

The SDWA differs substantially from previous Federal legislation in that it is directly applicable to all public water systems, not just those serving common carriers engaged in interstate commerce (U.S. Congress 1974).

b. Regulation. In responding to the mandate of the SDWA, USEPA has established a Drinking Water Program (DWP) composed of two major elements: the Public Water System Supervision (PWS) and the Groundwater Protection (GWP) programs. The former is designed to ensure that utilities comply with appropriate water quality standards, and the latter seeks to protect present and future sources of drinking water from contamination via underground injection wells. The principal regulatory mechanism of the DWP is the National Primary Drinking Water Regulations (NPDWR). The SDWA makes it clear that the responsibility for enforcing the NPDWR should ideally lie with the states and that the principal roles of USEPA are standard setting, supervision, and coordination. In fact, the 1996 reauthorization of the SDWA requires USEPA to publish operator certification guidelines for community and nontransient noncommunity public water systems.

3-3. 1996 Reauthorization of SDWA

a. The reauthorization updates the standards setting process. Originally, USEPA was required to provide 25 new standards every 3 years. This has been replaced with a new process based on occurrence, relative risk, and cost-benefit considerations. The USEPA is required to select at least five new candidate contaminants to consider for regulation every 5 years, but the regulation must be geared toward contaminants posing the greatest health risks. Additionally, the reauthorization requires states to develop operator certification programs or risk losing a significant portion (20 percent) of their revolving fund grant.

b. The USEPA is also required to identify technologies that are more affordable for small systems to comply with drinking water regulations. Small System Technical Assistance Centers are to be authorized to meet training and technical needs of small systems. States are to be given specific authority to grant variances for compliance with drinking water regulations for systems serving 3,300 or fewer persons and, with the concurrence of the USEPA Administrator, for systems serving more than 3,300 persons but fewer than 10,000 persons. Generally, it is not recommended that waivers be applied for at Corps projects; but if a need should arise, opinion of the Office of Counsel should be obtained prior to submittal.

3-4. The National Primary Drinking Water Regulations

a. General. These regulations specify the maximum permissible levels of contaminants that may be present in drinking water. While their principal purpose is to protect the public health, these regulations authorize a maximum contaminant level (MCL) to be set for a given substance, or group of related substances, even though no direct linkage to public health has been conclusively shown. The USEPA Administrator is empowered by the SDWA to consider economic feasibility as well as public health in establishing MCLs. The SDWA allows the Administrator of USEPA to establish treatment methodology criteria in order to provide general protection against a contaminant or group of contaminants without specifying any MCL.

b. Nomenclature. Several terms used in the SDWA and the DWP are specifically defined therein. Those that are especially pertinent are defined below.

(1) Contaminant. A contaminant is any physical, chemical, biological, or radiological substance or matter in water.

(2) Maximum contaminant level (MCL). The MCL is the maximum permissible level of a contaminant in water delivered to the free-flowing outlet of the ultimate user of a public water system, except in the case of turbidity where the maximum permissible level is measured at the point of entry to the distribution system. Contaminants added to the water under circumstances controlled by the user, except those resulting from corrosion of piping and plumbing caused by water quality, are excluded from this definition.

(3) Public water system. A public water system is a system for the provision to the public of piped water for human consumption, if such a system has at least 15 service connections or regularly serves an average of at least 25 individuals at least 60 days out of the year. Collection, pretreatment storage, treatment, storage, and distribution facilities are included in this definition. A public water system may be further classified as a “community” or “noncommunity” water system (Craun 1981).

(4) Community water system. A community water system is a public water system that serves at least 15 service connections used by year-round residents or regularly serves at least 25 year-round residents (Craun 1981; USEPA 1979c) .

(5) Noncommunity water system. Any public water system that is not a community water system is defined as a noncommunity water system (Craun 1981).

(6) Nontransient Noncommunity Water System (NTNC). A public water system that is not a community water system and that regularly serves at least 25 of the same individuals at least 6 months per year is on NTNC. Many Corps facility water systems are regulated in this classification. Individuals might include park rangers, resource administrative personnel, lock and dam operators, powerhouse personnel, and contract maintenance personnel among others.

(7) Transient noncommunity water system. This is a public water supply serving a transient population of at least 25 people a day at least 60 days a year. This may include parks, campgrounds, marinas, restaurants, and rest areas.

(8) Best available technologies (BAT). BAT is the technology referenced when USEPA sets the MCLs. The SDWA requires that the MCL be set as close as possible to the maximum contaminated level goal (MCLG) “with the use of the best technology, treatment techniques, and other means the Administrator finds available (taking cost into consideration).” Therefore, the MCL is generally affected by the technology available to remove that contaminant, because the MCL is set with cost of removal taken into consideration.

(9) Maximum contaminant level goal. The MCLG for each contaminant is a nonenforceable, health-based goal, set at a level at which no known or anticipated adverse effect on human health occurs. It allows for an adequate margin of safey, without regard to the cost of reaching these goals.

c. Coverage. The Drinking Water Regulations (DWR) apply to all public water systems except those meeting all of the following conditions:

(1) The system consists only of distribution and storage facilities (i.e., has no collection and treatment facilities).

(2) The system obtains all its water from, but is not owned or operated by, a public water system to which the regulations do apply.

(3) The system does not sell water to any person (individual, corporation, company, association, partnership, state, municipality, or Federal agency).

(4) The system is not a carrier that conveys passengers in interstate commerce.

Therefore, it is obvious that almost all water systems serving the public may be classified as public water systems and, thus, are regulated by the DWR. Facilities at Corps recreation areas, campgrounds, resorts, highway rest areas, and similar locations may frequently, however, be defined as noncommunity water systems. This is an important distinction since not all MCLs apply to such systems.

d. Maximum contaminant levels (MCLs).

(1) General comments. The MCLs are based on an assumed daily intake of water, or water-based fluids, of 2ℓ and are designed to protect the public from potential health effects of long-term exposure. Since these levels are not generally necessary to protect transients or intermittent users, many of the MCLs are not applicable to noncommunity water systems. An exception is nitrate, which is known to have an adverse effect on susceptible infants in even a short period of time. MCLs have not been developed for contaminants about which little is known, or which are only very rarely found in water supplies. However, the National Academy of Sciences and USEPA have developed Suggested No Adverse Response Levels (SNARLs) for several potential contaminants. SNARLs (also known as Health Advisories) are neither legally enforceable standards, nor directly comparable to MCLs since they have been developed for short-term, rather than lifetime, exposures. However, as more information comes available, it is likely that additional MCLs will be issued. Therefore, current SNARLs may be of some interest to water system designers and operations personnel, especially with respect to systems that will serve only transient populations. The SNARLs are most useful to managers and operators who must deal with such emergency situations as chemical spills, or industrial and agricultural accidents. Because of their very nature, the SNARLs are being continuously reviewed and revised; thus, they are not presented herein. Up-to-date information concerning them may be obtained from the Office of Drinking Water, U.S. Environmental Protection Agency, Washington, DC 20460. The current Internet address for the Office of Drinking Water is www.epa.gov/ow/. This Web site contains current published USEPA drinking water MCL's.

(2) MCLs for inorganic chemicals. The MCLs for inorganic chemicals (shown in Table 3-1) apply to all community and NTNC water systems. However, only nitrate limits require direct adherence for NTNC systems. If other contaminants exceed MCLs for the NTNC systems, an investigation of the possible health risk will be made. If it is determined that a health risk does exist, the MCL for that particular contaminant will apply. Thus, most Corps recreational area water systems would be subject to only the nitrate MCL. Compliance should be based on the analysis and sampling method as approved by the USEPA and/or the host state or territory as appropriate.

(3) MCLs for organic chemicals. MCLs for organic chemicals are presented in Table 3-1. They are applicable to community and NTNC water systems. Generally these chemicals would not be of regulatory interest for Corps recreation area water systems. However, all of the organic standards apply to noncommunity water systems if after an MCL is determined to be exceeded an investigation determines a risk to the public health exists.

(4) MCLs for total trihalomethanes. The MCL for total trihalomethanes is applicable to community systems serving a population of 10,000 or more and which add a disinfectant (oxidant) to the water during any part of the treatment process and community and NTNC systems obtaining water in whole or in part from a surface supply source. Compliance is determined on the basis of the running average of quarterly samples.

(5) MCLs for turbidity. MCLs for turbidity are applicable to both community and noncommunity water systems using surface water sources in whole or in part. In general terms, compliance with the turbidity MCL is based upon the monthly average of samples taken and analyzed daily at "representative entry points to the distribution system." The MCL for turbidity is based on a performance standard and should be 0.5 turbidity unit (TU) or less, but not to exceed 1.0 turbidity unit any time for surface water. Groundwater sources can be 5.0 TU or less, but not to exceed 15 TU at any time.

(6) MCLs for microbiological contaminants. MCLs for microbiological contaminants are applicable to both community and noncommunity water systems. Compliance is determined based on the analysis of samples taken at regular time intervals, and in numbers proportionate to the population served by the system. As of the August 1996 reauthorization of the SDWA, regulated standards for microbial contaminants included requirements from three regulations: Surface Water Treatment Rule (SWTR); Enhanced Surface Water Treatment Rule (ESWTR); and the Total Coliform Rule (TCR). Specifically, SWTR provides BAT requirements for *Giardia lamblia*, heterotrophic bacteria, and viruses. At the time of publication for this manual, the ESWTR proposed criteria guidelines for *Cryptosporidium*. Once the TCR was finalized, January 1991, criteria were in place to establish treatment techniques to achieve acceptable bacterial removal of fecal coliforms, total coliforms, and *E. coli*.

(7) MCLs for radioactivity. MCLs for radioactivity are rather complex and are generally based on limiting the annual dose to the whole body, or to any single organ. Basic requirements are presented in Table 3-1. The USEPA's proposed rule for radionuclides was published in July 1991.

Table 3-1
EPA Drinking Water Standards (USEPA)

Contaminant	Regulation	Status	MCLG, mg/L	MCL, mg/L
		Microbials		
Cryptosporidium	ESWTR	Proposed	0	TT
E. coli	TCR	Final	0	[1]
Fecal coliforms	TCR	Final	0	TT
Giardia lamblia	SWTR	Final	0	TT
Heterotrophic bacteria	SWTR	Final[2]	-	TT
Legionella	SWTR	Final[2]	0	TT
Total coliforms	TCR	Final	0	[1]
Turbidity	SWTR	Final	-	PS
Viruses	SWTR	Final[2]	0	TT
		Inorganics		
Antimony	Phase V	Final	0.006	0.006
Arsenic	Interim	Final	NA	0.05
Asbestos (fibers/1>10 µm)	Phase II	Final	7 million fibers per liter	7 MFL
Barium	Phase II	Final	2.00	2.00
Beryllium	Phase V	Final	0.004	0.004
Bromate	D/DBP (Disinfectants/ Disinfection-By-Product Rule)	Proposed	0	0.01
Cadmium	Phase II	Final	0.005	0.005
Chlorite	D/DBP	Proposed	0.08	1.0
Chromium (total)	Phase II	Final	0.10	0.10
Copper	LCR (Lead and Copper Rule)	Final	1.30	TT
Cyanide	Phase V	Final	0.20	0.20
Fluoride	Fluoride	Final	4.00	4.00
Lead	LCR	Final	0	TT
Mercury	Phase II	Final	0.002	0.002
Nickel	Phase V	Final	0.10	0.10
Nitrate (as N)	Phase II	Final	10.0	10.0
Nitrite (as N)	Phase II	Final	1.0	1.0

(Sheet 1 of 4)

Note: Standards are subject to change and the USEPA and host state should be contacted for up-to-date information. Abbreviations used in this table: NA - not applicable; PS - performance standard 0.5-1.0 ntu; TT - treatment technique.
[1] No more than 5 percent of the samples per month may be positive. (For systems collecting fewer than 40 samples per month, no more than 1 sample per month may be positive.)
[2] Final for systems using surface water; also being considered for groundwater systems.

Table 3-1. (Continued)

Contaminant	Regulation	Status	MCLG, mg/L	MCL, mg/L
		Inorganics (continued)		
Nitrite & Nitrate (as N)	Phase II	Final	10.0	10.0
Selenium	Phase II	Final	0.05	0.05
Thallium	Phase V	Final	0.0005	0.002
		Organics		
Acrylamide	Phase II	Final	0	TT
Alachlor	Phase II	Final	0	0.002
Aldicarb	Phase II	Final	0.001	0.003
Aldicarb sulfone	Phase II	Final	0.001	0.002
Aldicarb sulfoxide	Phase II	Final	0.001	0.004
Atrazine	Phase II	Final	0.003	0.003
Benzene	Phase I	Final	0	0.005
Benzo(a)pyrene	Phase V	Final	0	0.0002
Bromodichloromethane	D/DBP	Proposed	0	NA
Bromoform	D/DBP	Proposed	0	NA
Carbofuran	Phase II	Final	0.04	0.04
Carbon tetrachloride	Phase I	Final	0	0.005
Chloral hydrate	D/DBP	Proposed	0.04	TT
Chlordane	Phase II	Final	0	0.002
Chloroform	D/DBP	Proposed	0	NA
2,4-D	Phase II	Final	0.07	0.07
Dalapon	Phase V	Final	0.2	0.2
Di(2-ethylhexyl) adipate	Phase V	Final	0.4	0.4
Di(2-ethylhexyl) phthalate	Phase V	Final	0	0.006
Dibromochloromethane	D/DBP	Proposed	0.06	NA
Dibromochloropropane (DBCP)	Phase II	Final	0	0.0002
Dichloroacetic acid	D/DBP	Proposed	0	NA
p-Dichlorobenzene	Phase I	Final	0.075	0.075
o-Dichlorobenzene	Phase II	Final	0.6	0.6
1,2-Dichloroethane	Phase I	Final	0	0.005
1,1-Dichloroethylane	Phase I	Final	0.007	0.007
cis-1,2-Dichloroethylene	Phase II	Final	0.07	0.07

(Sheet 2 of 4)

Table 3-1. (Continued)

Contaminant	Regulation	Status	MCLG, mg/L	MCL, mg/L
		Organics (continued)		
trans-1,2-Dichloroethylene	Phase II	Final	0.1	0.1
Dichloromethane (methylene chloride)	Phase V	Final	0	0.005
1,2-Dichloropropane	Phase II	Final	0	0.005
Dinoseb	Phase V	Final	0.007	0.007
Diquat	Phase V	Final	0.02	0.02
Endothall	Phase V	Final	0.1	0.1
Endrin	Phase V	Final	0.002	0.002
Epichlorohydrin	Phase II	Final	0	TT
Ethylbenzene	Phase II	Final	0.7	0.7
Ethylene dibromide (EDB)	Phase II	Final	0	0.00005
Glyphosate	Phase V	Final	0.7	0.7
Haloacetic acids[3]	D/DBP	-	-	-
(Sum of 5; HAA5)	Stage 1	Proposed	-	0.06
-	Stage 2	Proposed	-	0.03
Heptachlor	Phase II	Final	0	0.0004
Heptachlor epoxide	Phase II	Final	0	0.0002
Hexachlorobenzene	Phase V	Final	0	0.001
Hexachlorocyclopentadiene	Phase V	Final	0.05	0.05
Lindane	Phase II	Final	0.0002	0.0002
Methoxychlor	Phase II	Final	0.04	0.04
Monochlorobenzene	Phase II	Final	0.1	0.1
Oxamyl (vydate)	Phase V	Final	0.2	0.2
Pentachlorophenol	Phase II	Final	0	0.001
Picloram	Phase V	Final	0.5	0.5
Polychlorinated byphenyls (PCBs)	Phase II	Final	0	0.0005
Simazine	Phase V	Final	0.004	0.004
Styrene	Phase II	Final	0.1	0.1
2,3,7,8-TCDD (dioxin)	Phase V	Final	0	0.00000003
Tetrachloroethylene	Phase II	Final	0	0.005
Toluene	Phase II	Final	1.0	1.0
Toxaphene	Phase II	Final	0	0.003

(Sheet 3 of 4)

[3] The sum of the concentrations of mono-, di-, and trichloracetic acids and mono- and dibromoacetic acids.

Table 3-1. (Concluded)

Contaminant	Regulation	Status	MCLG, mg/L	MCL, mg/L
		Organics (continued)		
2,4,5-TP (silvex)	Phase II	Final	0.05	0.05
Trichloroacetic acid	D/DBP	Proposed	0.3	NA
1,2,4-Trichlorobenzene	Phase V	Final	0.07	0.07
1,1,1-Trichloroethane	Phase I	Final	0.2	0.2
1,1,2-Trichloroethane	Phase V	Final	0.003	0.005
Trichloroethylene	Phase I	Final	0	0.005
Trihalomethanes[4]	Interim	Final	NA	0.1
(sum of 4)	D/DBP	-	-	-
-	Stage 1	Proposed	NA	0.08
-	Stage 2	Proposed	NA	0.04
Vinyl chloride	Phase I	Final	0	0.002
Xylenes (total)	Phase II	Final	10.0	10.0
		Radionuclides		
Beta-particle and	Interim	Final	-	4 mrem
photon emitters	R (Radionuclide Rule)	Proposed	0	4 mrem
Alpha emitters	Interim	Final	-	15 pCi/L
-	R	Proposed	0	15 pCi/L
Radium 226+228	Interim	Final	-	5 pCi/L
Radium 226	R	Proposed	0	20 pCi/L
Radium 228	R	Proposed	0	20 pCi/L
Radon	R	Proposed	0	300 pCi/L
Uranium	R	Proposed	0	20 µg/L

(Sheet 4 of 4)

[4] The sum of the concentrations of bromodichloromethane, dibromochloromethane, tribromomethane, and trichloromethane.

Subsequently, there was much controversy over the proposed radon standard. As of 1996, USEPA has chosen to delay promulgation of any radionuclides rule package since the development of a radon standard was an interrelated part of that package.

3-5. The National Secondary Drinking Water Regulations

a. General. USEPA has promulgated secondary as well as primary drinking water regulations (USEPA 1979b). The major difference between the two is that the secondary regulations are not enforceable at the Federal level. The regulations are intended to serve as guidelines for the states, but may be adopted as part of the drinking water program of any given state and, hence, become enforceable at that level. The purpose of the regulation is to guide the states in controlling contaminants that affect primarily the aesthetic qualities relating to public acceptance of drinking water. However, some of the contaminants may have health implications at higher concentration levels.

b. Secondary maximum contaminant levels (SMCLs). SMCLs for public water systems are presented in Table 3-2. Contaminants added to the water under circumstances controlled by the user, except those resulting from corrosion of piping and plumbing caused by water quality, are excluded. The SMCLs are designed to represent reasonable goals for

Table 3-2
Secondary Maximum Contaminant Levels (USEPA)

Contaminant	SMCL
Aluminum	0.05 + 0.2 mg/L
Chloride	250 mg/L
Color	15 color units
Copper	1.0 mg/L
Corrosivity	Noncorrosive
Fluoride	2.0 mg/L
Foaming Agents	0.5 mg/L
Iron	0.3 mg/L
Manganese	0.05 mg/L
Odor	3 TON[1]
pH	6.5 - 8.5
Silver	0.10 mg/L
Sulfate	250 mg/L
Total Dissolved Solids	500 mg/L
Zinc	5 mg/L

[1] Threshold odor number.

drinking water quality. They are important, though not federally enforceable, since undesirable aesthetic qualities may encourage users to rely on some alternative source (spring, cistern, etc.) that may be unsafe. Thus, every effort should be made, within the constraints of technological and economic feasibility, to produce water that meets the requirements of the secondary regulations.

3-6. Other Regulatory Requirements

a. Federal. A complete discussion of all Federal regulations that may impact on water system design and operation is beyond the scope of this manual. However, it is important for the planner/designer to understand that the SDWA is not the only Federal law that affects water systems. Other Federal legislation with provisions that may affect water systems would include the following (among others):

(1) Clean Water Act.

(2) Resources Conservation and Recovery Act.

(3) Hazardous Materials Transportation Act.

(4) Occupational Safety and Health Act.

(5) National Energy Conservation Policy Act.

(6) River and Harbor Act of 1899.

The effects of these, and other, Federal acts on water system design are minimal. In the vast majority of cases, compliance with the applicable state regulations will ensure compliance with pertinent Federal regulations as well.

b. State and local. All the states and many localities have legislation and regulations that affect the design of water supply systems either directly or indirectly. A review of all such requirements is clearly beyond the scope of this work. Fortunately, following the requirements of the state public health agency usually ensures that any local water quality problems will be minimal. To avoid possible conflicts, it is well worthwhile to contact state and local public health officials very early in the planning stage of project development. This is good practice even though Federal facilities may, in many cases, technically be exempt from state and local regulation. State and local problems often develop simply because the affected agencies are not consulted regularly and kept informed, and not because of any real conflict over technical issues.

3-7. Water Quality and Public Health

a. Introduction. Although ancient people did have some appreciation for the relationship between sanitation and public health, widespread treatment of public water supplies has developed only since the 1850's. Most historians point to the British cholera epidemics of 1845-1849 and 1853 as landmark events. In the latter case at least 69 of a total of nearly 11,000 deaths were attributed to a single well (in the Saint James Parish district of London), which was found to be polluted via a pipe draining a nearby cesspool. From that point, water treatment for the control and prevention of waterborne disease became more and more important. Today the emphasis in water treatment is changing somewhat, and while the control and prevention of the traditional diseases is still a concern, the possibly deleterious effects of literally thousands of chemical contaminants that may be present in drinking water supplies must be considered. A brief discussion of these problems is presented below.

b. Waterborne disease.

(1) General. Absolutely no natural water should be assumed to be free of microbial life. Some of these organisms (the pathogens) cause disease, some cause nuisance problems such as tastes and/or odors, and the vast majority are really of no particular consequence unless present in very great

numbers. Modern water treatment practice calls for the removal or inactivation of all organisms that may cause disease (this process is often called disinfection), but not necessarily the removal or inactivation of all life forms (sterilization). Organisms of special interest include bacteria, algae, fungi, molds, and viruses.

(2) Bacteria. Some waterborne diseases that may be traced to bacterial origin are noted in Table 3-3. Other bacteria, although not pathogenic themselves, can lead indirectly to disease by rendering water so aesthetically unpleasing that users turn to alternative, but unsafe, supplies such as polluted springs. Examples would include the iron bacteria frequently responsible for "red water" problems, and bacteria producing unpleasant tastes and odors. As a general rule, disinfection practices will control waterborne bacterial diseases. Therefore, special attention should be given to the design of disinfection facilities.

(3) Other organisms.

(a) Algae. Algae are nuisance organisms that may occasionally "bloom" (a bloom is defined as more than 1,000,000 cells/mL) and cause operational problems such as filter clogging as well as undesirable tastes and odors. Some algae produce toxic metabolites, but freshwater algae are not known to cause any waterborne diseases.

(b) Viruses. Viruses are the smallest of all the infectious agents that may be found in drinking water. They are probably not consistently removed to any great extent during conventional water treatment, but the methods used to detect and quantify them are so difficult and unreliable that the matter is open to debate. It is theoretically possible for virtually any enteric virus to be transmitted via drinking water and produce disease. However, only polio and hepatitis have been shown to do so. As a rule of thumb, outbreaks of waterborne diseases that cannot be traced to other causes are generally blamed on viruses.

(c) Protozoans. Protozoans are microscopic animals that may frequently be found in water. While many species have been identified, only three, *Entamoeba histolytica*, *Giardia lamblia*, and *Cryptosporidium* are of really major pathogenic significance. The first is the cause of amoebic dysentery (which can be a very serious condition) and is infectious only during the cyst stage. The cysts are quite resistant to chlorination, but fortunately are so large (8-12 micrometers) that they are readily removed by coagulation, flocculation, and sedimentation followed by granular media filtration. *Giardia* cause a recurring form of diarrhea frequently called giardiasis. *Giardia* cysts are also relatively large and are adequately removed in the manner described above. *Cryptosporidium* cysts are more difficult to remove than *Giardia* cysts. But when operated properly, a treatment plant producing finished water turbidity with less than 0.1-0.2 nephelometric turbidity units (NTU) through conventional treatment or direct filtration can usually achieve adequate treatment. Waterborne disease outbreaks in Milwaukee, WI (March 1993), Racine, WI (March 1994), and Washington, DC (December 1993) were all attributed to *Cryptosporidium*, contributing to the generation of additional treatment regulations. Other animals, such as the parasitic worms (nematodes, trematodes, and cestodes), may be found in water but are likewise adequately removed via conventional practices.

(4) Indicator organisms. The direct examination of drinking water for all possible pathogenic organisms is impractical for a number of reasons, for example:

(a) There are a wide variety of pathogens.

(b) Many pathogenic organisms may be present in very small numbers (e.g., viruses) and thus may escape detection; and analytical procedures for the isolation, identification, and enumeration of many pathogens are difficult, unreliable, time-consuming, and/or very expensive.

Thus, public health officials have long sought the elusive "ideal indicator" organism. Such an organism would have the following characteristics:

(a) Indicate the presence of pathogens in both raw and treated water.

(b) Be somewhat more hearty than pathogens.

(c) Be present in biologically contaminated waters in great numbers (certainly in greater numbers than the pathogens).

(d) Be readily identifiable via simple, quick, inexpensive, straightforward analytical procedures.

(e) Be such that the population density of the indicator is directly related to the degree of contamination.

Needless to say, no such organism has been found. However, over many years public health professionals in the United States have come to depend on the coliform bacteria to serve this purpose. They are not a perfect indicator, but their presence in treated water is ample reason to suspect the microbiological safety of the water. Unfortunately, the mere absence of coliforms does not ensure that water is free from pathogens. For many years the "coliform" count was the specific tool used to evaluate quality. Now, however, discoveries indicate that high levels of heterotrophic bacteria may result in false negative samples for total coliform. Problems with such indicators and the lack of ability to readily

Table 3-3
Some Bacterial Waterborne Diseases

Disease	Responsible Organism	Comment
Cholera	*Vibrio cholera*	Very serious. Organism can survive in clean or turbid water.
Salmonellosis	Several species of *Salmonella*	Range from typhoid fever (*S. Typhosa*) to "ptomaine poisoning."
Shigellosis	Several species of *Shigella*	Common cause of acute diarrhea. *S. Dysenteriae* causes bacillary dysentery.
Leptospirosis	Several species of *Leptospira*	Comparatively uncommon, but worldwide.
Tularemia	*Francisella tularensis*	Extremely virulent organism. Can survive in water for long periods.
Tuberculosis	*Mycobacterium tuberculosis*	Very resistant to chlorination.
Montezuma's Revenge	Variants of *Esherichia coli*	Generally harmless to natives, but not visitors.
Gastroenteritis	Many bacteria, e.g., *Yersina enterocolitica*	Survives in very cold waters. Also caused by other types of organisms.

measure and detect these and other organisms have led EPA to establish accepted treatment techniques.

c. Chemical hazards.

(1) Current situation. In recent years, the water supply industry, regulatory agencies, consumer advocates, lawmakers, and the general public have become increasingly aware of, and concerned about, the presence of various chemicals, some of them quite exotic, in public water supplies. During this period, analytical capabilities have advanced at an almost incredible pace while the knowledge needed to interpret the resulting data has developed comparatively slowly. Thus, the industry is in the unfortunate position of being able to detect the presence of contaminants, especially metals and organic compounds, to the ppb or mg/L level or lower, but has virtually no rational basis on which to assess the public health consequences of the vast majority of the substances so identified. This is especially true with regard to long-term effects of low-level exposures.

(2) Outlook. It seems highly likely that at least some of the compounds now being detected in water will prove to be deleterious to health, even in very low concentrations, and that such compounds will continue to be discovered. The reauthorized SDWA (August 1996) requires USEPA to select at least five new candidate contaminants to consider for regulation every 5 years based on the contaminants posing the greatest health risk. Resulting regulation is to be developed from a balance of occurrence, relative risk, and cost-benefit considerations. Since treatment techniques for the removal of low levels of contaminants often tend to be rather complicated and expensive, it behooves planners and designers of small water supply systems to consider alternate sources of water very carefully. The expenses associated with investigatory items such as test wells and complete laboratory analyses may seem almost prohibitive; but they must be compared to those associated with major renovations, process additions, more sophisticated operation, or shifting to a new water source after collection, treatment, and distribution facilities are in place. If there is any reason to believe that a potential water source may be contaminated, great caution should be used in developing that source. Planners and designers must certainly look beyond current water quality regulations, although to do so admittedly involves as much art as science. Cost consideration for monitoring and compliance must always be included in economic comparisons.

3-8. Contaminants Found in Water Supplies

a. Definition. The word contaminant is subject to varying usage. In this manual the term is applied to any physical, chemical, biological, or radiological substance found in water. Thus, a contaminant is not necessarily good or bad. The term pollutant is sometimes applied to identify a contaminant that has a deleterious effect.

b. Occurrence. The number of contaminants that may be present in a water supply is virtually unlimited. Whenever substances listed in Table 3-1 are found in concentrations greater than those shown, the water should be viewed with caution and possible alternative sources should be investigated. However, it should be understood that only a few of the contaminants shown warrant outright rejection of the supply. It is important to understand that, where the principal concern is aesthetics, regional factors are very important. For example, hardness levels that are perfectly acceptable in one geographical area might be grounds for rejection of the supply in some other location. Fortunately, only a few contaminants are of general interest. A brief discussion of some of the more common contaminants and important properties of water is presented below. For more information the reader is directed to the references listed in Appendix A.

c. Turbidity.

(1) Definition. Turbidity results from optical properties that cause light to be scattered and/or absorbed rather than transmitted directly through the medium of interest. Turbidity in water is caused by the presence of suspended matter such as clay, silt, algae, or bacteria (i.e., any finely divided organic or inorganic matter). The suspended materials that cause turbidity are considered undesirable since they may represent a direct or indirect hazard to public health and certainly render water aesthetically unpleasing. As a general rule, turbidity is measured by nephelometry (i.e., measurement of the portion of a light beam that is scattered in some selected direction-usually 1.57 radians (rad) (90 degrees (deg)) to the direction of the light path) and reported in NTUs. Occasionally other methods and reporting units may be used.

(2) Occurrence and removal. Provisions must almost always be made to remove turbidity when surface waters are to be used for public water supply. While plain sedimentation is of some value for pretreatment, it is generally ineffective as a sole means of treatment. This is true because the particles that usually contribute most of the turbidity are of colloidal size (1 to 200 nm in diameter). These particles are so small that their behavior is controlled by their state of hydration (interaction with water molecules) and surface electrical charges (similar particles develop similar charges and thus repel each other electrically) rather than by gravitational effects. In the typical surface water treatment plant, coagulants or flocculants are added to interact with the colloidal particles to coalesce into larger particles under the influence of gentle mixing. Filter alum, a hydrated form of aluminum sulfate, is by far the most commonly used coagulant in the United States. These larger particles are then typically removed by sedimentation and granular media filtration. By these means, water that is sparkling clear (0.1 NTU) can consistently be produced. The diversity of materials makes it impractical to define any meaningful maximum recommended turbidity level for raw surface waters. In fact, the difficulty encountered in turbidity removal is often inversely proportional to the initial turbidity. For the most part, turbidity requirements are regulated under the SWTR and the TCR.

d. Color. "True" color is caused by the presence of any of a number of dissolved materials and, unlike turbidity, is more likely to occur in ground waters than surface waters. "Apparent" color includes true color plus the effects of any suspended substances that may be present. This latter component is easily removed along with turbidity. Color in water supplies usually results from the presence of such factors as metallic ions, humic substances, industrial wastes, or algae, and is usually more pronounced at higher pH. Color, per se, is not a public health problem, although some substances that can impart color to water are hazardous. Therefore, when color is encountered in a water supply, it is important to determine the cause. It is best to avoid potential water sources that exhibit significant color. However, if a suitable alternative is not available, color removal should be seriously considered. The specific process selected will vary with the source of the color, but chemical oxidation and adsorption have both been effective in some cases whereas ordinary water treatment is generally ineffective against true color. Even a slight bit of color is so aesthetically displeasing to some people that they will prefer to use colorless water from a source of questionable sanitary quality (e.g., a spring).

e. Tastes and odors. Tastes and odors in water generally result from the presence of algal, bacterial, or actinomycete metabolites; decomposing organic matter; or dissolved gases, although industrial wastes are occasionally implicated. As is the case with color, difficulties with tastes and odors are usually more related to aesthetics than to public health. Taste and odor problems are especially difficult to deal with since they tend to be intermittent (e.g., they may follow the growth cycles of the responsible organisms). This is compounded by the fact that even very minute (ppb level) concentrations of some substances can be detected by many people. Thus, for example, it might be possible to remove 90 percent or more of some given odorant without significantly reducing complaints from customers. Therefore, when a choice is available, water sources known to be free of tastes and odors are much to be preferred. Taste and odor problems vary considerably, and when removal is to be practiced, some care should be exercised in the selection of a method. Aeration, chemical oxidation (e.g., with potassium permanganate), and activated carbon adsorption have been effective in a number of installations.

f. Hardness.

(1) General. Hardness in water is caused by the presence of divalent metal ions. While a number of these can occur, solubility constraints are such that only calcium (Ca^{++}) and magnesium (Mg^{++}) are generally present to a significant extent in natural waters. Hardness may be a problem for both surface and ground waters, but is more likely in the latter case. No definitive relationship (either positive or negative) has been established between hardness in drinking water and public health; however, it may be very deleterious from an economic

and aesthetic point of view. Excessive hardness creates a high soap demand and thus makes bathing difficult, interferes with laundry and other washing activities, contributes to deterioration of fabrics, and promotes excessive deposition of calcium carbonate ($CaCO_3$) and magnesium hydroxide ($Mg(OH)_2$) on pipes, especially hot water pipes and boiler tubes. On the other hand, insufficient hardness interferes with rinsing operations and promotes rapid corrosion of metallic waterlines and appurtenances. The optimal total hardness of a given water supply is a function of many factors, but is generally between 50 and 80 mg/L as $CaCO_3$. Magnesium hardness greater than 40 mg/L as $CaCO_3$ is very undesirable in hot water applications.

(2) Classification and removal. There are no hard and fast rules as to exactly what constitutes hard water, but the values shown in Table 3-4 are widely accepted in the United States. As a general rule, water with a total hardness greater than about 125 mg/L as $CaCO_3$ (or magnesium hardness greater than about 40 mg/L as $CaCO_3$) should be softened prior to use if it is operationally and economically feasible to do so. Chemical precipitation and ion exchange are both effective. The former is often less costly, but the latter is far simpler and is, therefore, usually preferred for small installations.

Table 3-4
Classification of Hardness in Water (from Dufor and Becker 1968)

Hardness, mg/L as $CaCO_3$	Classification
0-20	Soft
20-60	Slightly Hard
60-120	Moderately Hard
120-180	Hard
Above 180	Very Hard

g. Iron.

(1) Occurrence. Iron may be found in both surface and ground waters, but is more commonly a problem in the latter. In water exposed to the atmosphere, ferrous iron (Fe++) is readily oxidized to ferric iron (Fe+++) by oxygen, and various relatively insoluble precipitates are formed. Thus, surface waters containing sufficient soluble iron to cause significant problems are fairly rare. An exception is water in the hypolimnion of a stratified reservoir. In such an environment molecular oxygen is not readily available and insoluble ferric iron may be reduced to the soluble ferrous form. The acceptable limit for drinking water is 0.3 mg/L.

(2) Problems. Iron problems usually occur when the soluble ferrous form present in the raw water is oxidized to the insoluble form in the distribution system or after delivery to the user. Since the precipitates are colored (yellowish, reddish, or brownish), they are immediately obvious to customers and, therefore, constitute a color problem. In addition they can produce a metallic taste, stain plumbing fixtures, and interfere with laundry and cleaning operations. Similar problems result when corrosive water is supplied through iron or steel pipes. A related phenomenon involves certain attached autotrophic bacteria, such as *Crenothrix* and *Gallionella*, that may establish residence in distribution systems. These organisms derive energy from the oxidation of iron and store the resultant precipitates in cellular material. Occasionally “clumps” of the bacteria break away from pipe walls or pumps and cause periodic problems. Iron can be sequestered by various “corrosion inhibitors” such as polyphosphates, or may be removed from water by ion exchange/adsorption or by a combination of oxidation, sedimentation, and filtration. The latter process is widely used, with oxygen, chlorine, and potassium permanganate all finding substantial usage as the oxidant.

h. Manganese. Manganese is less common than iron, but causes similar problems (the characteristic color is dark brown or black). Manganese chemistry is complex, but removal methods are similar to those previously described for iron. One significant difference is that manganese oxidizes in air at a very slow rate and hence may be somewhat more likely to be present to a significant extent in surface waters. The acceptable limit for manganese in drinking water is 0.05 mg/L.

i. Alkalinity. Alkalinity may be defined as the ability of water to neutralize an acid, and is determined by titration against a known standard acid (usually 0.02 N sulfuric acid). Alkalinity has traditionally been reported in terms of mg/L as $CaCO_3$. This is somewhat confusing nomenclature since the chemical species responsible for virtually all the alkalinity of natural waters is the bicarbonate ion (HCO_3^-). The optimal amount of alkalinity for a given water is a function of several factors including pH, hardness, and the concentrations of dissolved oxygen and carbon dioxide that may be present. As a general rule, 30 to 100 mg/L as $CaCO_3$ is desirable although up to 500 mg/L may be acceptable. Alkalinity is apparently unrelated to public health (at least directly), but is very important in pH control. Alum, gaseous chlorine, and other chemicals occasionally used in water treatment act as acids and, therefore, tend to depress pH. Alkalinity resists this change and thereby provides buffer capacity. Many waters are deficient in natural alkalinity and must be supplemented with lime (CaO or $Ca(OH)_2$) or some other chemical to maintain the pH in the desirable range (usually 6.5 to 8.5). Alkalinity values can change significantly for groundwater between samples taken at the wellhead and samples taken from a storage reservoir that are a few hours old.

j. pH. pH is especially important with respect to body chemistry, the effectiveness and efficiency of certain water treatment processes, and corrosion control. Most natural waters have a pH in the range of 6.5 to 8.5. Since this range is generally acceptable, pH control usually requires making only relatively minor adjustments rather than wholesale changes. Exceptions arise when low-alkalinity waters must be treated with acidic chemicals such as alum or chlorine gas, with waters that have been softened by the lime-soda process, or with well waters that are supersaturated with carbon dioxide and hence may have a very low pH (down to about 4.5). The occurrence of pH lower than about 4 to 4.5 is indicative of the presence of mineral acids and, hence, possible contamination by industrial wastes.

Chapter 4
Water Quantity Requirements

4-1. Introduction

The United States is blessed with an abundant supply of both ground and surface waters. Unfortunately, the population is not distributed in the same pattern as are the water resources, and the hydrologic cycle does not operate at steady-state. As a result, local water shortages have already occurred in most parts of the country and may be expected to increase in frequency and occur in other areas in the coming years. Thus, increasing importance is being attached to preparation of water use projections and the planning necessary to ensure that water demands are met in a manner that is both timely and cost-effective. In this chapter these problems are addressed specifically with respect to water supply systems serving municipal and rural communities, military installations, recreation areas, and highway rest areas. In addition, consideration is given to water conservation and its effect on water supply system design. Although the principal thrust of this manual is toward small systems, some discussion of municipal water supply system design is necessary to present the pertinent design information in logical fashion.

4-2. General Considerations

a. Water use rate variation. Water supply system design is complicated to a considerable extent by the fact that water use rates are influenced by a number of factors. For example, municipal use might be affected by some, or perhaps all, of the following:

(1) Climate.

(2) Standard of living.

(3) Extent of sewerage.

(4) Extent of metering.

(5) Price of water.

(6) Season of the year.

(7) Day of the week.

(8) Time of day.

(9) Special events.

(10) Firefighting requirements.

(11) Commercial development.

(12) Industrial development.

(13) Landscape irrigation.

(14) Water quality.

(15) Availability of alternate supplies.

(16) Distribution system pressure.

(17) System maintenance and management.

(18) Real or potential water shortages.

(19) Legal constraints.

The list is not intended to be all-inclusive, nor are all the factors presented independent. However, it is sufficient to make the point that for any given water supply system, many variables can affect water use. Thus, no single water use rate can be used to design every system or even every component of a given system. Specific water use rates that may generally be considered to be important include the following:

(1) Average annual use.

(2) Average monthly use.

(3) Maximum monthly use.

(4) Average weekly use.

(5) Maximum weekly use.

(6) Average daily use.

(7) Maximum daily use.

(8) Maximum hourly use.

(9) Maximum instantaneous use.

For specialized systems, for example those serving recreation areas or highway rest areas, other use rates may also be important. Examples include average weekend use and maximum weekend use. The magnitudes of use variations that may be expected for various types of water supply systems are considered in subsequent sections of this chapter.

b. Average use. A measure of average water use, such as the average daily use, is needed to determine if the yield of a water source is sufficient to safely supply water over long periods of time and to determine the storage capacity needed to assure that an adequate supply is available during critical periods (e.g., droughts).

c. Peak use. A measure of peak use, such as the maximum hourly use, maximum instantaneous use, or fire flow is needed to size distribution facilities (e.g., pipelines, booster pumps, storage) so that peak demands can be satisfied without overtaxing production and treatment facilities or causing excessive pressure losses.

d. Intermediate use. A measure of use between the average and peak values is ordinarily used in the hydraulic design of treatment facilities. Many engineers design treatment processes to operate normally at the average daily flow rate, but be hydraulically capable of passing a greater flow, say the maximum daily flow. This occasional "overloading" or "overrating" of the plant, or portions thereof (e.g., rapid sand filters), may be acceptable even though effluent quality is reduced to some extent. Alternatively, the plant may be designed to operate without overloading at the maximum daily use rate. In this situation, the plant may normally operate at process rates lower than those used in design, or various treatment units may be taken off line and held in reserve until needed. The latter approach is frequently used, especially with rapid sand filters. Another possibility is that the treatment plant may be designed to meet average demands by operating for only a portion of the day. Higher rates of demand can then be met rather easily by extending the hours of operation. This approach is usually uneconomical for larger cities, but can be very attractive for small operations.

4-3. Storage Requirements

a. Introduction. Depending upon the particular situation, several different types of storage facilities may be needed to ensure that an adequate water supply is always available. Examples include raw water storage (e.g., surface water impoundment), finished water storage at the treatment plant (e.g., clear well and backwash tank), and distribution storage (e.g., ground, elevated or hydropneumatic tanks). Regardless of the type of facility, the basic method used to determine the required storage volume is essentially the same.

b. Raw water storage.

(1) General. Where a surface water supply is used, it may be possible to design a supply system to operate without any raw water storage facility dedicated specifically to water supply. Examples might be a small town drawing water from a large multipurpose impoundment, or even a large city taking water from one of the Great Lakes. However, in the general case, some provision must be made to catch water during periods of moderate to high streamflow and store it for later use. The size of the storage facility required is usually determined based upon consideration of hydrologic information such as minimum dry-weather streamflow, average streamflow and rainfall/runoff patterns, and some average measure of water use, for example, the average daily use. The mass diagram, or Rippl, method has traditionally been used to determine storage requirements. This technique is amenable to either a simple graphical or more complex analytical approach, and is widely known since it is covered in many standard water supply and applied hydrology textbooks (Clark, Viessman, and Hammer 1977; Fair, Geyer, and Okun 1966a; Linaweaver, Geyer, and Wolff 1966; Salvato 1982; Steel and McGhee 1979). Essentially the same method is used to size equalization basins used in wastewater treatment (Metcalf and Eddy 1991). The mass diagram technique is very flexible and may be used in either a deterministic or probabilistic format. For more information the reader is directed to the references noted above.

(2) Design criteria. In the eastern United States, raw water reservoirs are usually designed to refill every year. In more arid regions, streamflow is less dependable and water must be stored during wet years for use during extended dry periods. Typical American practice over the last 50 or 60 years has been to size raw water storage facilities to be adequate to compensate for any drought condition expected to occur more often than once in about 20 years, plus some additional reserve storage allocation (e.g., 25 percent). This rule of thumb, combined with the implementation of use reduction measures when reservoir storage is depleted to some critical level, ordinarily results in a reasonable trade-off between storage requirements and user inconvenience. However, in recent years many other methods have appeared in the water supply literature. Regardless of the method used, it is important to consider the effects of evaporation, seepage, and siltation any time a reservoir is to be designed.

(3) Groundwater. When groundwater serves as the source of supply, no provision for long-term raw water storage is usually made. Short-term storage is, however, often useful. A good example is a situation where groundwater is extracted by a number of relatively low-yield wells (i.e., low-yield water supply to total water demand), pumped to a central storage tank and then withdrawn for distribution. This technique is especially useful for equalizing pumping rates when water from some, or all, of the wells requires treatment prior to distribution. The mass diagram approach mentioned in *b*(l) above may be used to size the storage tank so long as the inflow and outflow rates are known.

c. Finished water storage. Distribution storage facilities are used to meet peak demands (including fire flows), allow continued service when the supply is interrupted, equalize system pressures, eliminate continuous pumping, and facilitate the use of economical pipe sizes. While it is possible to size tanks using the mass diagram approach, it is more common to rely on various rules of thumb. Salvato (1982) suggests that, depending upon system size and type, distribution storage volume may vary from about one-half the average daily use, to the maximum daily use, to a 2- or 3-day supply. Even when rule-of-thumb criteria are used to size distribution storage facilities, it may be useful to conduct a mass diagram type of analysis (*b*(l) above) to ensure that peak demands can be met. Storage requirements for filter backwash tanks, clear wells, and other reservoirs can also be determined from mass diagrams if so desired.

4-4. Municipal Water Use

a. Introduction. As previously mentioned (paragraph 4-2*a*), municipal water use varies widely from city to city and from time to time for a given city. American Water Works Association (AWWA) (1975, 1981) and U.S. Geological Survey (1975) present data that indicate clearly that U.S. water use patterns vary considerably with geographical location. This point is further emphasized by the per capita water use data contained in Metcalf and Eddy (1991), Murray and Reeves (1972), and van der Leeder (1975).

b. Design approach. Design values for water use rates are usually determined as follows:

- Select the design period.
- Forecast the population to be served by the end of the design period.
- Estimate the expected average water use rate at the end of the design period.
- Estimate design use rates by multiplying the average use rate by selected factors.
- Determine the required fire demand from insurance requirements.
- From the various use rates calculated above, select those applicable to various system components.

A brief discussion of each step is outlined below. The same basic format is followed in later sections where rural, recreation area, military installation, and highway rest area systems are specifically addressed.

(1) Design period. As a general rule, the design period for portions of the system that may be readily enlarged (e.g., well fields and treatment plants) is chosen as 10 to 25 years. Components that are difficult and costly to enlarge (e.g., large dams) may be designed for a longer period, say 25 to 50 years. Prevailing interest rates are an important factor, with higher rates generally favoring shorter periods. The source of funds is also important. When funding assistance is available (e.g., in the form of grants or subsidized loans) there is a tendency to overdesign. In effect, this represents extension of the design period. Water lines serving residential areas are usually sized for full development since residential requirements in developing areas tend to change rapidly and replacement of such lines is costly.

(2) Population forecasts. Population forecasts are usually based on some combination of official census data; special studies made by various private and public interests (e.g., market surveys); the attitudes of local people (especially business and political leaders) toward expansion; and input from state, regional, and local planning agencies. Most states have developed population forecasting formulas that are adjustable for various regions within the given state. Because population forecasting has long been of interest to sanitary engineers, the topic is adequately covered in most standard water supply and wastewater engineering texts (Clark, Viessman, and Hammer 1977; Technical Manual 5-813-3; Fair, Geyer, and Okun 1966a; Metcalf and Eddy 1991; Steel and McGhee 1979).

(3) Average per capita use. Average per capita water use is usually determined from past experience in the local area or similar areas, regulatory agency requirements, or the water supply literature. Many studies of municipal water use have been reported and an overall average of about 450 to 800 liters per capita per day (L/cd) (100 to 175 gallons per capita per day (gpcd)) seems to be applicable for the United States. Publications prepared by the AWWA, U.S. Geological Survey and others (Metcalf and Eddy (1991), Murray and Reeves (1972), and van der Leeder (1975)) indicate an estimated national average of 755 L/cd (166 gpcd) for 1975. However, the reported range of values (less than 227 L/cd (50 gpcd) to more than 2273 L/cd (500 gpcd)) is so wide that specific knowledge about the area to be served should take precedence over national, or even regional, averages. A substantial improvement in water use forecasting can be realized by disaggregating municipal water use as described below.

(4) Disaggregated use. Municipal water use can be disaggregated (if sufficient data are available) and allocated to various water use sectors. An example scheme is shown in Table 4-1. Many other arrangements could, of course, be used. Typical allocations expressed as percentages of the average daily use are shown in Table 4-2. Disaggregation generally improves forecasting accuracy since the effects of such factors as climate (i.e., need for irrigation), commercial activity, industrial development, and water conservation programs can be readily considered. Residential water use can be further

Table 4-1
Scheme for Disaggregating Municipal Water Use Using Municipal Water Use Sectors

- Residential
  - Single-family
    - Interior
    - Exterior
  - Multiple-family
    - Interior
    - Exterior
- Commercial
  - Interior
  - Exterior
- Industrial
  - Process
  - Cooling
  - Sanitary
- Public and Institutional
  - Interior
  - Exterior
  - Hydrant Flow
- Unaccounted-for
  - Metering Error
  - Loss

disaggregated as shown in Table 4-3 (interior use only) and Table 4-4. A frequency distribution graph (USEPA 1980) indicates the frequency with which various average daily residential water use rates may be expected to be exceeded. Limited data will often preclude the complete disaggregation of water use. However, if at all possible, disaggregation should proceed at least to the level of separating residential, commercial, and industrial use. In regions where lawn watering is practiced, every effort should be made to consider residential interior and exterior use separately. This latter category can account for as much as 80 percent of afternoon residential use during a summer drought and thus has a great effect on peak as well as average use.

(5) Other water use rates.

(a) Regardless of the method used to determine the average water use (i.e., per capita estimation or disaggregation by sector), it is common to apply multipliers (factors) to the value selected to estimate other use rates. Some of these multipliers are shown in Table 4-5. The range of values indicates that significant differences exist between systems. As a general rule, the ratio of peak to average use rate increases with

Table 4-2
Disaggregated Municipal Water Use as Percentage of Average Daily Use

<table>
<tr><th rowspan="2">Reference[1]</th><th colspan="5">Use Sector</th><th rowspan="2">Average Daily Use (gpcd)[2]</th></tr>
<tr><th>Residential</th><th>Commercial</th><th>Industrial</th><th>Public</th><th>Unaccounted-for</th></tr>
<tr><td>Linaweaver, Geyer, and Wolff 1966</td><td>41</td><td>18</td><td>24</td><td colspan="2">17</td><td>-</td></tr>
<tr><td>California Department of Water Resources 1976</td><td>68</td><td>10</td><td>18</td><td colspan="2">4</td><td>-</td></tr>
<tr><td>Murray and Reeves 1972</td><td>38</td><td colspan="2">32</td><td colspan="2">30</td><td>166</td></tr>
<tr><td>AWWA 1975</td><td>42</td><td>18</td><td>22</td><td colspan="2">18</td><td>179</td></tr>
<tr><td>Deb 1978</td><td>52</td><td>17</td><td>15</td><td>7</td><td>9</td><td>153</td></tr>
<tr><td>Deb 1978[3]</td><td>39</td><td>12</td><td>31</td><td>5</td><td>13</td><td>162</td></tr>
<tr><td>Deb 1978[4]</td><td>40</td><td>15</td><td>25</td><td>5</td><td>15</td><td>160</td></tr>
<tr><td>Frey, Gamble, and Sauerlender 1975</td><td>49</td><td>12</td><td>21</td><td colspan="2">18</td><td>166</td></tr>
<tr><td>Fair, Geyer, and Okun 1966a</td><td>33</td><td colspan="2">43</td><td>7</td><td>17</td><td>150</td></tr>
<tr><td>Steel and McGhee 1979[5]</td><td>44</td><td>15</td><td>24</td><td>9</td><td>8</td><td>177</td></tr>
</table>

[1] Entries in this column are included in Appendix A.
[2] Gallons per capita per day. To convert to liters per capita per day, multiply by 3.7854.
[3] Average of 27 Pennsylvania utilities.
[4] 1978 national average.
[5] Projected for 2000 AD.

Table 4-3
Disaggregated Interior Residential Water Use as Percentage of Average Daily Interior Residential Use

Reference[1]	Use Sector: Toilet Flushing	Bathing	Laundry	Dishwashing and Cooking	Drinking	Miscellaneous	Average Daily Use (gpcd)[2]
Linaweaver, Geyer, and Wolff 1966	30	35	20	15			-
California Department of Water Resources 1976	42	32	14	12			-
Deb 1978	40	30	15	6	5	4	60
Dufor and Becker 1962	41	37	7	11		4	-
Bailey and Wallman 1971	39	34	14	11		2	64
U.S. Environmental Protection Agency 1980	35	20	22	23			46
U.S. Environmental Protection Agency 1981	40	30		25		5	65

[1] Entries in this column are included in Appendix A.
[2] Gallons per capita per day. To convert to liters per capita per day, multiply by 3.7854.

Table 4-4
Disaggregated Residential Water Use as Percentage of Average Daily Residential Use

Reference[1]	Use Sector: Interior	Exterior	Average Daily Use (gpcd)[2]
Linaweaver, Geyer, and Wolff 1966	77	26	80
California Department of Water Resources 1976	56	44	-
Bailey et al. 1969	93	7	-
Dufor and Becker 1962	96	4	55
Deb 1978	94	6	64

[1] Entries in this column are included in Appendix A.
[2] Gallons per capita per day. To convert to liters per capita per day, multiply by 3.7854.

decreasing system size and increasing use of water for lawn watering.

(b) Residential water use and water use rates have been studied by a number of researchers. However, many water supply textbooks rely heavily on the results of a project undertaken for the Federal Housing Administration by Johns Hopkins University during the 1960's. Reporting on this project, Linaweaver, Geyer, and Wolff (1966) presented mathematical relationships that may be used to estimate average residential water use for metered and sewered areas for any period of interest. The basic expression in SI units is

$$\bar{Q} = \bar{Q}_d + 6010(a)(\bar{L}_s)(\bar{E}_{pot} - \bar{P}_{eff}) \text{ with } \bar{Q} > \bar{Q}_d \qquad (4\text{-}1)$$

where

$\bar{Q}$ = expected average demand for any period (liters per day)

$\bar{Q}_d$ = expected average residential use for periods of a day or longer (liters per day)

a = number of dwelling units considered

$\bar{L}_s$ = average irrigable area per dwelling unit (hectares)

$\bar{E}_{pot}$ = estimated average potential evapotranspiration for the period in question (millimeters of water per day)

$\bar{P}_{eff}$ = amount of natural precipitation effective in satisfying evapotranspiration and thus reducing the need for lawn watering (millimeters of water per day)

$\bar{Q}_d$ may be estimated as a function of the average market value of the dwelling units as follows:

Table 4-5
Relative Water Use Rates

	Use Rate					
Reference[1]	Average Monthly	Average Daily	Average Maximum Monthly	Average Maximum Weekly	Maximum Daily	Maximum Hourly
Salvato 1982	1	-	1.5	-	2.25	4.5
Salvato 1982	1		1.5		4.5	9
Salvato 1982	-	1	-	-	4	9.5
Salvato 1982	-	1	-	-	-	6
Steel and McGhee 1979	-	1	1.28	1.48	1.8	2.7
Alabama State Board of Health 1978	-	1	-	-	1.5	2.25
Fair, Geyer, and Okun 1966a	-	1	-	-	1.5	2.5
Clark, Viessman, and Hammer 1977	-	1	-	-	1.35	3.4
Clark, Viessman, and Hammer 1977	-	1	-	-	2.9	6.1
Clark, Viessman, and Hammer 1977	-	1	-	-	4.1	9.1
Clark, Viessman, and Hammer 1977	-	1	-	-	4.2	12.1
Metcalf and Eddy 1991	-	1	1.2	1.4	1.8	-

[1] Entries in this column are included in Appendix A.

$$\bar{Q}_d = 594 + 13.1\ (V) \qquad (4\text{-}2)$$

where V is the average market value of the dwelling units ($1,000's) corresponding to the year 1963. Clark, Viessman, and Hammer (1977) suggest that this method is still valid if property values are deflated to 1963 conditions using local indices. During high demand periods precipitation becomes negligible and Equation 4-1 reduces to:

$$\bar{Q} = \bar{Q}_d + 6010\ (a)\ (\bar{L}_s)\ (\bar{E}_{pot}) \qquad (4\text{-}3)$$

The estimated average potential evapotranspiration may be estimated from climatological data. However, in a study of some 41 residential areas scattered over the United States, Linaweaver, Geyer, and Wolff (1966) found an average value of 7.11 millimeters (mm) (0.28 inches (in.)) of water per day. They also developed a series of design curves that may be used to estimate maximum daily and peak hourly water use rates based on housing density (dwelling units per acre) and the number of dwelling units served. These curves are reproduced in some water supply texts (Clark, Viessman, and Hammer 1977).

(6) Fire flows. The volume of water used annually for fighting fires in a typical municipality is ordinarily very small compared to the total use. However, short-term fire demands can be very high and in many cases govern the design of distribution facilities. Fire flow requirements are usually based on the recommendations of insurance industry groups (Insurance Services Office), and for residential areas generally range from 30 to 500 liters per second (L/s) (500 to 8000 gallons per minute (gpm)) depending upon the population served. For the central business district of large cities, the fire flow requirement may be as much as 760 L/s (12 000 gpm) for a single fire plus an additional 500 L/s (8000 gpm) for a second fire. The duration for which these flows must be maintained varies from 4 to 10 hours depending upon the size of the community. If a given system is incapable of delivering the recommended fire flow, fire insurance rates are adjusted upwards. As a general rule, it is assumed that the system must be able to deliver the fire flow concurrently with the maximum daily demand at a pressure of not less than 138 kilopascals (kPa) (20 pounds (force) per square inch). Thus, it is not surprising that the fire condition often controls distribution system design.

c. Commercial and industrial use. Industrial and commercial water use should be estimated separately if possible and then added to other disaggregated uses to reach an estimate of total municipal use. Furthermore, if sufficient information is available, individual, industrial and commercial users should be considered separately. Unfortunately, it is very difficult to predict what the water use of a given industrial or commercial establishment will be without very specific data. However, such estimates are often based on average use rates since data on individual operations are ordinarily not available. Some general guidance is presented in several sources (McGauhey 1968; Metcalf and Eddy 1972; Planning and Management Consultants, Inc., 1980b; Salvato 1960, 1982). Kollar and MacAuley (1980) have presented a rather detailed analysis of industrial water requirements.

4-5. Rural Water Use

a. Introduction. Design criteria that are appropriate for larger municipal water systems are often quite inappropriate for smaller community water systems. Generally, the average per capita water use for small community water systems (especially rural systems) is less than the average per capita residential water use for large cities. This is not universally true, however, since cultural factors, property values, the extent of lawn watering, and many other variables may influence use for a given community. Occasionally small community water systems serve commercial and/or industrial users, which have a major impact on facility design.

b. Design approach.

(1) Special considerations.

(a) For municipal systems, the peak water use rate considered in the design of distribution facilities is usually either the maximum hourly demand or the combination of the fire flow and the maximum daily demand. While design is frequently controlled by the latter case, such is not always true. However, the diversity of customers served, the grid-system layout of distribution piping, and the use of a 150- or 200-mm- (6- or 8-in.) diameter minimum pipe size combine to make consideration of urban residential demands for periods of less than one hour generally unnecessary.

(b) The population served by rural water systems tends to be rather disperse (i.e., low areal population density) with two to five service connections per mile of pipe being fairly typical. Thus, rural systems must be designed from a somewhat different perspective than municipal systems. Typically, fire protection to the extent recommended by the insurance industry is uneconomical, piping systems must be of the branching rather than the grid type, and the minimum pipe size is quite small (say 50-mm (2-in.) diameter). Unfortunately regulatory agencies have not always recognized these differences.

(c) Because in larger high-density residential areas and municipalities the extra costs associated with providing fire flow capacity are spread over many customers, the price of water service is not affected to a significant degree. The economics of rural systems are, however, entirely different and generalized fire protection is usually completely infeasible. Of course it may be argued that any dependable public water supply offers some fire protection. And, in some special cases, it may be economical to provide standard fire flows to a small area located near the water source or a major distribution point (e.g., elevated tank).

(d) Widespread development of rural water systems has occurred only during the recent past. Perhaps the two most important factors leading to this growth have been governmental assistance programs, primarily those of the Farmers Home Administration (FmHA), and the development and acceptance of polyvinyl chloride (PVC) pipe. A third factor has been the willingness of some state regulatory agencies to relax their design criteria somewhat to accommodate rural needs.

(2) Average water use.

(a) A limited number of controlled studies of rural water use have appeared in the literature. In most cases, average rural residential water use has been found to be somewhat less than average urban residential use. At least two factors would seem to be important in interpreting this finding. One is that many rural families have historically been less dependent on high-water-use appliances than have urban families. This is partly the result of economic factors and partly the result of the fact that rural areas are generally unsewered. Cultural differences are also probably significant in this regard. A second, though related, factor is that the unit price of rural water is generally higher than that of urban water. Several authors have suggested that rural water demand is rather price elastic. For example, data from a study of some 150 rural water systems in Kentucky (Grunwald et al. 1975) was used to develop the following expression in English units:

$$Q = 7.57\ P^{-0.92} \tag{4-4}$$

where

Q = average monthly water use, per dwelling unit

P = unit price of water, dollars per 1,000 gallons

The magnitude of the exponent on P makes it obvious that price is indeed an important factor. Hughes and Israelsen (1976) compared this expression to data obtained for a number of small water systems, mainly in the western United States, and found a similar trend. However, their data indicated that the coefficient on P might be somewhat low. It is likely that this results from increased irrigation use for the western systems.

(b) When all factors are considered, it seems quite reasonable to assume that rural residential water use will eventually approach the urban value. An abundant supply of high-quality water for domestic use is a major determinant of the quality of life in rural areas. Agricultural and industrial demands for water must also be met. Therefore, for design purposes, average residential water use rates for small rural water systems may be taken as equal to average residential use rates for nearby urban areas. Such an approach will almost certainly be conservative, especially for unsewered areas or areas not previously served by a public water system. Goodwin and Doeksen (1984) reported that data collected for 660 observations in Oklahoma indicated the following equation (English units) provided the best statistical reliability and economic consistency:

$$\begin{aligned} Q_m = & -1505.73 + 954.86N + 33.85Y + 102.76E \\ & +55.49C + 183.60H + 953.86G \\ & +2221.92I \end{aligned} \tag{4-5}$$

where

Q_m = average monthly water use per customer, gallons

N = number of persons in the household

Y = year the house was built

E = total years of education for household head

C = number of cattle watered

H = number of horses watered

G = garden, dummy variable where G = 1 if garden is watered and G = 0 if no.

I = income, dummy variable where I = 1 if income exceeds \$40,000 and I = 0 if \$40,000 or less

As a general rule a reasonable degree of conservation in this regard will not be excessively expensive since costs of the system components most directly affected (e.g., transmission piping, raw water storage, and treatment facilities) are less related to flow rates (especially for small flows) than are costs of other components such as distribution piping.

(c) Some care must be used in the selection of design flow rates for small rural water systems since it may not be desirable to operate treatment facilities on a 24-hour-per-day basis. This situation arises, in part, because many of the capital costs associated with small treatment facilities are relatively unrelated to facility capacity. For example, Hansen, Gumerman, and Culp (1979) have reported typical complete package surface water treatment plant costs. These costs are exclusive of raw water intake and pumping facilities, clear well storage, high service pumping, land, and site work, except for foundation preparation. The costs indicate that, for the lower flow rates, the differential price paid for extra plant capacity is relatively small. For example, capacity can be doubled from 15 to 30 L/min (4 to 8 gpm) (at a filtration rate of 80 L/min per square meter (2 gpm per ft^2)) for an additional investment of only about 14 percent. If a shift to high rate filtration is acceptable, capacity can be increased by a factor of five for essentially the same incremental cost.

(d) An additional factor to consider is that small system operating costs tend to be dominated by operator salaries. Therefore, it is often economical to produce all the treated water needed on a typical day during a relatively short period, say 4 to 8 hours. This approach results in savings in the salaries of operating personnel, provides ample time for routine maintenance, often does not increase debt service to a significant extent ((c) above), and allows the flexibility to produce extra water occasionally by simply extending operation for an hour or so. As the average demand increases, plant "capacity" can be increased without additional capital expenditure by gradually lengthening the normal operating day. Thus, especially when a surface water source is used, the combination of a larger than necessary treatment plant and reduced operating time can be very attractive. Groundwater supplies frequently do not need treatment other than disinfection and, therefore, generally require less operator control. Thus the foregoing argument may not be valid for systems relying on wells or springs.

(e) If the decision is made to produce water for only a fraction of a day, raw water storage, pumping, transmission, and treatment, and finished water transmission, storage, and pumping facilities must all be designed accordingly.

(3) Peak water use.

(a) Since fire flows are not usually considered in the design of rural water systems, some other measure of peak water use must be used. The similarity of the customers normally served (i.e., mostly residential) is such that many designers and regulatory agency personnel feel that the maximum instantaneous demand should be used to size distribution facilities.

(b) It is obvious that for a given portion of a typical rural water system, maximum (peak) instantaneous demand should be a function of the number of customers served. Furthermore, it would certainly seem reasonable to assume that as the number of customers served increases, the ratio of peak instantaneous demand to customers served should decrease. That is, as the number of customers served increases, it becomes increasingly unlikely that all customers will demand water at the maximum rate simultaneously. Thus, peak instantaneous residential water use may logically be estimated as the product of the number of water services, or connections, times some peak use rate per connection (which is a function of the number of connections).

(c) Representative relationships between peak instantaneous residential use per connection and the number of connections served are presented in Alabama State Board of Health (1978), Ginn, Corey, and Middlebrooks (1966), and Hughes and Israelson (1976). Obviously differences of opinion exist.

(d) Hughes and Israelson (1976) and Hughes and Caufield (1977) have reported FmHA claims that some 5,000 systems have been designed using minimal standards (e.g., 3.8 L/min (1 gpm)/connection for 100 or more connections) without apparent difficulty (i.e. without subsequent customer complaints). However, in these cases it seems highly likely that the flow to individual homes is occasionally limited by the hydraulic capacity of the distribution lines. On the other hand, the recommendations of some state regulatory agencies (e.g., Alabama State Board of Health 1978) seem overly conservative, especially when one considers the frictional losses associated with flow at, for example, 55 L/min (15 gpm) through a typical 20-mm- (3/4-in.-) diameter water service line and 15-mm- (½-in.-) diameter house piping system (about 12 m (40 ft) of water in a 30-m (100-ft) run of 20-mm- (3/4-in.-) diameter plastic service line alone).

(e) The conservative approach to design taken by many regulatory agencies stems directly from missions related primarily to protection of public health. Thus, design standards and criteria are adopted that ensure the integrity of a water supply system against hypothetical simultaneous events having a probability of occurrence very near zero. While this is in many ways an admirable attitude, it is so costly that the result may be that the rural population in a given area is forced to continue to rely on individual water supplies of questionable quality because a community supply system cannot be economically justified. Clearly a common-sense approach to balancing these and other potential risks is needed. This has been recognized by many states.

(f) Where agreeable to regulatory agency personnel, and in the absence of good local data, the design use (flow) rates shown in Figure 4-1 are suggested. According to Ginn, Corey, and Middlebrooks (1966), and Hughes and Canfield (1977) these use rates correspond to a return interval of approximately 27 years. That is, demands in excess of those shown may be expected to occur once in about 27 years. These criteria should be more than satisfactory from the viewpoint of rural customer satisfaction and provide ample protection of public health.

(4) Other water use rates. The lack of available data makes the estimation of rural water use rates at least as much an art as a science. Two use rates that have not been discussed, but that may have design significance, are the maximum daily and maximum monthly demands. After surveying the literature, Hughes and Israelson (1976) suggested that a maximum daily rate of about 2.3 L/min (0.6 gpm) per residential connection appeared reasonable. Their data also indicate that a ratio of maximum monthly use rate to average monthly use rate of around 1.5 should be sufficient for most design purposes. An alternative approach is to estimate peak daily and monthly use rates from the "peaking" factors discussed above in paragraph 4-4*b*(5). As previously discussed, the use of these factors is likely to overestimate rural demands, at least in the short run.

4-6. Recreation Area Water Use

a. Introduction. Water systems serving recreation areas are similar in some respects to rural community systems, but also differ in some respects. As a rule they are rather compact, have branching type distribution piping, and must respond to widely varying water use rates that may be affected by many variables including the following:

(1) Location.

(2) Type of facilities provided.

(3) Visitation rates.

(4) Visitation patterns.

(5) Season of the year.

(6) Day of the week.

(7) Special events.

(8) Irrigation requirements.

(9) Weather conditions.

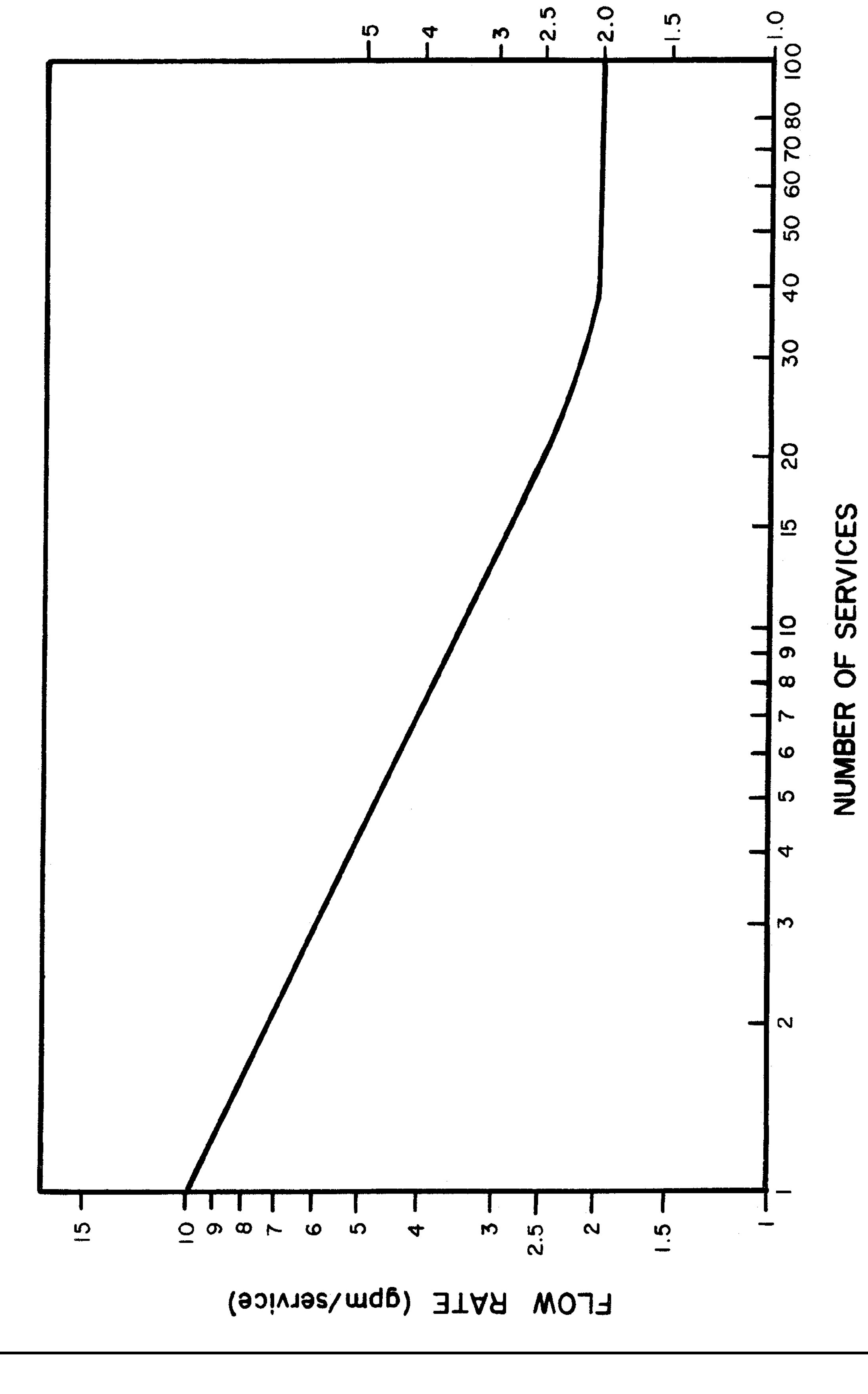

Figure 4-1. Instantaneous peak design flow rates (to convert gpm to liters/min, multiply by 3.7854)

b. Design approach.

(1) Corps guidance.

(a) It is suggested that average water use at Corps facilities be estimated as the sum of 110 to 190 L/day (30 to 50 gpd) for each day-shift employee (night-shift employees are generally neglected), 570 L/day (150 gpd) for each dwelling, 20 L/day (5 gpd) for each visitor expected to use flush-type toilets, plus any additional requirements (e.g., cooling water or lawn watering). It is further suggested that peak demands be based on a combination of 190 L/day (50 gpd) per dwelling and "reasonable" assumptions as to maximum frequency of use of facilities. These and other values can be developed from U.S. Environmental Protection Agency publication EPA 570/9-91-004, May 1991, entitled "Manual of Individual and Non-Public Water Supply Systems." Typically, most designers obtain their minimum demand standards from state or local standards. Many states probably use these standards. Many of these drinking water systems will require environmental permitting for construction; hence, the use of state-approved standards is recommended.

(b) USEPA (1991) recommends the use of the average water use rates summarized in Table 4-6.

(c) Francingues and Green (1976) have reported a detailed study of water use at a typical Corps recreation area near Memphis, TN. They found that campsite occupancy varied widely (zero to 98 percent of the design value) and that the maximum average observed weekend occupancy (78 percent) occurred, as expected, on a holiday weekend. A typical weekend visitation consisted of an average of 4 persons per occupied campsite (the range was 2 to 6) for a 2-day period (the range was 1 to 3). Some 63 percent of all recreational vehicles observed were equipped with wastewater holding tanks, and 79 percent of those vehicles made use of the trailer dumping station. A summary of observed water use for the period between 23 May and 1 September is presented in California Department of Water Resources (1976). The average water use of 458 L (121 gallons) per occupied campsite per day compares favorably with the 450-L (119-gallon) figure reported by Matherly et al. (as cited in Francingues et al. 1975) for the Sullivan Access Area at Lake Shelbyville, IL. The reported average per capita use of 87 L/day (23 gpd) is somewhat lower than the 114 L/day (30 gpd) suggested in EM 1110-2-400. However, division of the observed 458 L/day (121 gpd) per occupied campsite by an average of 4 persons per camping party yields a per capita use rate of about 110 L/day (30 gpd). Therefore the recommended 110 L/day (30 gpd) seems very reasonable. Peak water use

Table 4-6
Water Requirements for Recreation Areas

Type of Facility	Gallons per day[1]
Bath houses (per bather)	10
Camps: Construction, semipermanent (per worker)	50
Day with no meals served (per camper)	15
Luxury (per camper)	100-150
Resorts, day and night, with limited plumbing (per camper)	50
Tourist with central bath and toilet facilities (per person)	35
Laundries, self-serviced (gallons per washing, i.e., per customer)	50
Parks: Overnight, with flush toilets (per camper)	25
Trailers with individual bath units, no sewer connections (per trailer)	25
Trailers with individual bath, connected to sewer (per person)	50
Picnic: With bathhouses, showers, and flush toilets (per picnicker)	20
With toilet facilities only (gallons per picnicker)	10

[1] To convert to liters per day, multiply by 3.7854.

rates were not reported; however, some inferences can be drawn from the wastewater generation data that were presented. The ratio of peak hour to average daily wastewater flow varied from about 1.8 to about 4.2. It is reasonable to assume a similar ratio for peak hour to average daily water use since consumptive use at recreation areas is typically small (say 15 percent or so).

(2) Peak water.

(a) The existing guidance summarized in the previous section (4-6*b*(1)) is sufficient to estimate average water use for most recreation areas. However, little guidance with respect to peak use rates is available in Corps publications.

(b) Peak water use rates suitable for design purposes may be determined by consideration of particular facilities to be provided together with an estimate of the maximum expected visitation (Corps recreation facilities are normally designed to be used to capacity within 3 years of construction). One method that has been used successfully in many applications is based on the concept of "fixture units" (Hunter 1941). Each fixture or group of fixtures is assigned a relative peak demand rate in terms of fixture units. The total peak demand is normally determined by summing the fixture unit values of all the fixtures to be provided and then consulting a design curve. Hunter (1941), Salvato (1982), and USEPA (1980) present the basic information needed to use this method. When fixtures are likely to impose a continuous demand, the continuous portion of the demand should be estimated separately and added to the total fixture demand. Fixture unit values for fixtures not shown may be assumed by comparison to a similar fixture. The fixture unit values shown are for the total demand. Where applicable the hot and cold demand may each be estimated as three-fourths of the value shown. Salvato (1982) has reported that such estimates tend to err somewhat on the high side. This is not altogether undesirable, especially in the design of washhouses where showers and water heaters are to be provided. In such cases, insufficient capacity to deliver water on demand during peak periods can result in scalding (as well as other injuries resulting from panic, falling, etc.) to those taking showers. Information such as that presented in Metcalf and Eddy (1972), Salvato (1982), and USEPA (1974) may also be useful in estimating peak flow at recreation facilities.

4-7. Rest Area Water Use

a. Introduction. Water systems supplying highway rest areas are very similar (though they are often more compact) to recreation area systems. Thus, much of the information presented above, especially in paragraph 4-6, should be directly applicable. Design information specifically developed for highway rest area systems has been collected, organized, and published by the Federal Highway Administration (Folks 1977).

b. Design approach.

(1) Water use rates. The Federal Highway Administration suggests (Folks 1977) that water requirements may be estimated by using average daily traffic volumes for the six peak weekends, assuming that 9 percent of the vehicles will stop, and assuming that 25.4 L (6.7 gallons) of water will be needed per vehicle. A design period of 20 years is used unless there is some specific reason to do otherwise. Thus, the 20-year projected traffic volume should be used in the calculation. Peak water use rates may be estimated by consideration of operating data from existing systems that are similar. Francingues et al. (1975) have reported that approximately 16 percent of the daily use can be expected to occur in a 1-hour period around midday and approximately 67 percent occurs in an 8-hour period from about 8 a.m. to 4 p.m. Of course, special events can alter the timing of peak demands. Additional sources of demand may include fire protection (usually not provided), irrigation (varies from 25 mm (1 in.) per week to 76 mm (3 in.) per week during season depending upon the climate and specific needs), and drinking and wash water (for wastewater tanks) needed for recreational vehicles.

(2) Other considerations.

(a) For new construction, low-water-use fixtures should be used where possible. This practice, when combined with minimizing irrigation needs and avoiding using water for aesthetic purposes (e.g., fountains), unless a plentiful supply is readily available, will reduce the amount of water required without adversely affecting the function of the rest area.

(b) Storage requirements may be determined by the mass diagram approach mentioned in paragraph 4-3*b*(1) once average and peak water use rates are known. It is recommended (Folks 1977) that storage capacity be provided even if it is not absolutely necessary since pumping and pipe costs are generally reduced by storage. A detailed analysis should be conducted to determine the best solution for each individual site.

4-8. Water Conservation

a. Introduction. In recent years conserving water and energy by reducing water use or loss has received increasing attention. Most research reported so far pertains to the implementation of various conservation measures during critical periods (i.e., droughts) and indicates that, at least in the short run, use rates can be reduced dramatically. The extent to which such reductions can be sustained when and/or where water is plentiful is not fully known. However, many flow reduction devices and low-flow fixtures and appliances are now available, and most manufacturers of fixtures and appliances are presently replacing their standard lines with low-use models. Thus, some tendency toward long-term reduction in water use may be expected. AWWA is actively promoting water conservation programs to improve the efficiency of utility operations and reduce the waste of water. Information on possible water conservation practices is presented in AWWA (1975).

b. Flow reduction measures. The USEPA (1981) has reported on a study of various flow reduction measures and has synthesized information from a number of sources. The results indicate that not all flow reduction measures are equally economical. Data gathered from several sources and published by AWWA (1975) indicate that the cost of the water saved varies by orders of magnitude (i.e., 0 to $5 per 3800 L (1000 gallons) saved). It should be noted that many conservation measures are interrelated. Thus, the flow reduction expected to result from the application of a combination of measures may be less than the sum of the individual reductions expected for each measure applied alone (e.g., a pressure-reducing valve on the service line and a low- flow shower head would have considerable interaction).

c. Design implications. From the viewpoint of the design engineer the effects of water conservation programs can be twofold. Firstly, total operation and maintenance costs may be reduced as less water must be supplied. Unfortunately, fixed costs may cause unit operation and maintenance costs to rise; thus, customers may not realize significant dollar savings. Secondly, it may be possible to reduce the capacity of various water supply facilities and/or delay system expansion as the result of conservation. For small specialized water supply systems (such as those serving campgrounds), it may be possible to analyze the effects of conservation by simply considering the implications of the flow reduction expected to result from the use of low-water-use devices. For other systems, the problem may be considerably more complex since existing facilities (not necessarily fitted with water-saving devices) are often responsible for a considerable fraction of the total water use. The Corps has developed and reported (Fair, Geyer, and Okun 1966a; Planning and Management Consultants, Ltd., 1980a, 1980b) a method for forecasting the effects of flow reduction on the design of water supply systems that is applicable to these general cases. Since different conservation measures affect different aspects of total water use, it is a good idea to disaggregate water use into as many sectors as is practicable. A possible scheme for disaggregation was presented in Table 4-1. When conservation is being considered, the effect on each sector should be determined independently and then summed to determine total effect.

Chapter 5
Water Sources

5-1. Introduction

a. General considerations. The selection of a water source may range from a relatively simple, straightforward choice dictated by local conditions to a complex and difficult decision involving the careful and deliberate consideration of many factors. At the very least, the following points should be considered for each available alternative source of supply:

(1) Adequacy and reliability with respect to providing water in sufficient quantity.

(2) Expected water quality.

(3) Development cost.

(4) Operation and maintenance cost.

(5) Monitoring and health requirements.

b. Alternative sources. Ordinarily, there are no more than four alternative categories of sources of supply to consider:

(1) Connection to an existing system.

(2) Water hauling.

(3) Development of groundwater resources.

(4) Development of surface water resources.

Of course there may be more than one alternative source within each category. For example, one may have the option of obtaining groundwater via wells in several locations or from springs, or of purchasing water from more than one existing system. Thus, it is theoretically possible to have several options. However, practicalities often limit the choices substantially. In any event, an important tool for the decision-making process is a sanitary survey of all alternative sources of supply. The conduct of such a survey and other important design elements are discussed in some detail in the following sections.

5-2. Sanitary Survey

a. Introduction. A sanitary survey should be performed for all alternative sources of supply. The validity of such a study is highly dependent upon the background and experience of the investigator. The services of a qualified sanitary or environmental engineer, sanitarian, or other public health professional should be obtained for this purpose.

b. Purposes. The principal purposes of a sanitary survey are to discover, investigate, and evaluate all conditions that might adversely affect the quality of a water supply or the adequacy of the supply to deliver water at a satisfactory rate. The survey also affords the opportunity to gather other basic information that may be useful in analyzing the general suitability of the source.

c. Sampling. The details of the survey will vary depending upon the source under study and prevailing local conditions. However, the collection of samples for subsequent chemical and physical analysis and microscopic and microbiological examination will always be a key element. This is no simple matter since the collection of truly representative samples is nearly always a challenge. Even when the physical constraints on sampling are minimal, it is easy to inadvertently contaminate them by faulty sampling technique and lack of attention to detail. This is especially true of samples to be subjected to microbiological examination for colifom organisms, or analysis for trace organic chemicals or trace elements. It is good practice to obtain detailed instructions and sampling procedures from laboratories that will be analyzing the samples. State and local health departments are excellent sources of information and often will supply sterile sample containers and analyze microbiological samples. Health departments usually maintain lists of approved laboratories where other analyses can be performed. All samples should be obtained, handled, processed, and analyzed in a manner conforming to American Public Health Association (1980), U.S. Environmental Protection Agency (USEPA) (1979a), or specific regulatory agency guidelines.

d. Analyses. The exact analyses to be performed are specified by either state, local, or Federal regulations generally depending on the finished system size and classification. Generally, in the absence of gross pollution, the list of analyses should include at least those itemized below:

(1) Acidity.

(2) Alkalinity.

(3) pH.

(4) Free carbon dioxide.

(5) Total residue.

(6) Total volatile residue.

(7) Total hardness.

(8) Calcium hardness.

(9) Temperature.

(10) Color.

(11) Taste.

(12) Odor.

(13) Turbidity.

(14) Nitrate nitrogen.

(15) Total chloride.

(16) Total fluoride.

(17) Total chlorine demand.

(18) Free available chlorine.

(19) Total coliforms.

(20) Fecal coliforms.

It is also good practice to have samples analyzed by some scanning-type methodology to identify the various organic compounds that may be present. This is expensive, but often worthwhile, especially if there is any reason to believe that such contaminants might be present.

e. Data interpretation. The interpretation of data generated by the water analyses must be closely coordinated with and based upon the results of other portions of the sanitary survey because the quality of water taken from different types of sources and under different conditions is naturally expected to vary. For example, water taken from an impounded river should not be judged by the same standards as water taken from a municipal distribution system. Thus, consultation with public health officials and other knowledgeable professionals is the first step in interpreting survey findings. General guidelines for evaluating the quality of water supplies were presented in Section 3-8. Other topics specifically related to various types of water sources are discussed in the following sections.

5-3. Existing Supplies

a. Introduction. Connection to an existing drinking water supply system is the source of choice if such can be accomplished economically. However, this decision should be based upon a careful evaluation of all available information rather than on the mere presence of an existing system.

b. Advantages.

(1) General. There are several potential advantages to tapping onto an existing water supply system. Some examples are listed below:

(a) Source development costs are avoided.

(b) Operation and maintenance are often greatly simplified.

(c) Substantial operation and maintenance costs may be avoided.

(d) Administrative responsibility may be greatly reduced.

(e) Regulatory burdens may be reduced or eliminated.

(f) Certain legal liabilities may be avoided.

(2) Operation and maintenance. While all these factors may be significant, those directly related to operation and maintenance are frequently the most important (USEPA 1979c). The most critical aspect is that small systems often cannot afford, nor do they really need, to employ highly qualified full-time water system managers and operators. The minimum level of operator qualification is specified by state, local, or Federal authorities. In evaluating the expense of any system, the cost to meet and maintain the required standard can be a heavily weighted factor. Thus, connection to an existing water system can be very attractive.

(3) Regulatory. Release from some or all of the regulatory burden of the SDWA may also be an important factor in certain situations. The NPDWR do not apply when the receiving system consists solely of storage and distribution facilities, all water is obtained from a publically owned system to which the regulations do apply, and the receiving system neither sells water nor is a carrier conveying passengers in interstate commerce. Obviously, when all these conditions are met, considerable expense and effort can be avoided. Community systems are generally precluded from taking advantage of this situation since they buy water for resale. Specialized systems, however, such as those serving rest stops and recreation areas may qualify. In theory, no major compromise with respect to water quality is involved since the water must meet or exceed the requirements of the SDWA when it enters the receiving system. The regulatory burden is merely shifted from the receiving system to the supplying system.

c. Disadvantages.

(1) General. There are also potential disadvantages to tapping onto an existing system. These may be described in broad terms as related to

(a) Management and operation.

(b) Connection costs.

(c) Water quality.

Each of these general areas is discussed below.

(2) Management and operation. One potential disadvantage is that the receiving system has virtually no control, and often little influence, over the management and operation of the supplying system. A second is that the receiving system is somewhat at the mercy of the supplying system with regard to the price paid for water. A third, and associated, potential disadvantage is that it may not be possible to negotiate a satisfactory long-term water purchase agreement. When a series of short-term agreements must be negotiated, there is always the possibility that the management of the supplying system may lose their desire to cooperate. This is especially true if the water demand in the supplying system's own service area increases to the point of taxing existing facilities. In this situation, one may expect a higher priority to be placed on meeting these local demands than selling water to other systems. Thus, it is evident that careful consideration must be given to short- and long-term effects of a water purchase agreement on both the receiving and the supplying systems.

(3) Connection costs. Major economic disadvantages may arise when the connecting pipeline must be long or pass through difficult terrain, the pressure at the connection point is low or highly variable, booster pumping is required, or substantial storage must be available to equalize the flow at the diversion point. While some of these conditions may require significant operation and maintenance expense and effort, their principal effects will be on initial capital investment. This may not always be a true disadvantage, however, since extra funds may be more readily available for initial investment than for continuing operation.

(4) Water quality.

(a) General. The existing system should be investigated in as much detail as possible during the sanitary survey phase of planning. Special emphasis should be placed on factors that might influence future water quality. It is a mistake to assume that water quality will always be acceptable simply because the supplying system must, by law, comply with state and Federal regulations.

(b) System operation and maintenance. The surveyor should look carefully at all intakes, pumping stations, treatment plants (including all operations and processes), storage facilities, distribution systems, and connections with other systems, especially industrial and fire protection systems. The system should be investigated for actual or potential sanitary defects such as direct and indirect cross connections, improper location of water mains, or broken or leaky mains. Since a complete onsite inspection of the entire distribution system is often impractical or prohibitively expensive, indirect evidence such as the existence of an aggressive cross connection control program, good maps of the system, administrative attention to detail, good record keeping, an adequate shop, and supply of spare parts and equipment may be important. Attention should be given to assessing the competence and dependability of operating and management personnel and the overall philosophy of system administrators.

d. Other considerations.

(1) Introduction. TM 5-813-1 suggests that the investigation of an existing supply include at least the following items.

(a) Source.

(b) Reliability.

(c) Quantity developed.

(d) Ultimate quantity.

(e) Excess supply available not already allocated.

(f) Type of treatment.

(g) Rates in gallons per minute at which supply is available.

(h) Cost per thousand gallons.

(i) Distance from site to existing supply.

(j) Variation in pressure at the point of diversion.

(k) Ground elevation at point of diversion and at point of use.

(l) Existence of contaminating influences.

A brief discussion of topics deserving special attention is presented below.

(2) Institutional arrangements. If the decision is made to connect to an existing system, all institutional arrangements should be made before the construction contract is let. This will aid in preventing unforeseen changes during construction that could affect the preferred water source alternative. In many projects, the Corps has paid the capital costs for construction with the larger system agreeing to accept ownership after construction and provide continued operation and maintenance of the system.

(3) The connection. The designer should be sure to specify that the actual connection between the systems be made in a readily accessible location, and that valves are placed so that the two systems can be quickly isolated from each other if necessary. The master meter should be located in a well-protected but accessible box or vault. The piping arrangement should be designed so that the meter can be bypassed easily when service is required. The possibility of backflow from the receiving system to the supplying system should be prevented in a way that will comply with the water surveyors regulations. The meter selected for use should be accurate over the expected range of flow rates. This is an important consideration since the pipeline will often be designed for a flow rate in excess of the actual flow rate, especially that for the early years of a project. Ordinarily, the actual connection and the meter installation will become the property of the supplying system. Thus, it is important that the interests of the receiving system be well protected by proper design.

e. Summary. Connecting to an existing supply system may substantially reduce routine operation and maintenance costs and effort; but water purchase costs, quality, and quantity are subject to control by the supplying system. Thus, the decision to tap onto an existing system should not be made lightly. On the whole, it appears that where economic factors are favorable, the advantages usually outweigh the disadvantages.

5-4. Groundwater

a. Introduction. When connection to an existing water supply system is not feasible or desirable, the development of groundwater resources is often the logical choice. This is especially true if the quality of the water is such that minimal treatment (and hence operator time and effort) is required. While groundwater may be obtained from springs, shallow wells, or deep wells, the emphasis in this manual is on deep wells. However, much of the basic material on deep wells is also applicable to shallow wells. Springs are usually not suitable for any but the very smallest systems, and the likelihood of finding a good spring in the right location is low. Readers interested in the development of springs are referred to American Association for Vocational Instructional Materials (1973), Cairncross and Feachem (1978), Folks (1977), Salvato (1982), USEPA (1979c), and U.S. General Accounting Office (1982).

b. Wells.

(1) Sanitary survey. When a well supply is being considered, information in addition to that described in Section 5-2 should be obtained as part of the sanitary survey. The following specific items deserve attention:

(a) Character of local geology.

(b) Slope of ground surface.

(c) Size of catchment area.

(d) Probable rate of recharge of water-bearing formations.

(e) Nature and type of soil and underlying strata.

(f) Depth to water table.

(g) Variations in depth to water table.

(h) Thickness and location of water-bearing strata.

(i) Location, log information, yield, and water quality analysis of nearby wells.

(j) Nature and location of sources of pollution.

(k) Possibility of surface water entering the supply directly.

(l) Influence of *any* surface water on the quality of the well water, indirectly.

(m) Physical, chemical, bacteriological, and radiological analyses of the raw water.

(n) Type of treatment required.

(o) Well spacing required to prevent mutual interference.

(p) Legal clearances required because of proximity to the wells of others.

(q) Drawdown data from nearby wells.

(r) Total seasonal and long-term pumpage from the area.

(s) Permeability of the aquifer.

(t) Velocity of groundwater flow.

(u) Rainfall amount, distribution, and intensity.

Much of this information may be available from state and local health departments, state geological agencies, the U.S. Geological Survey, local water utilities, well drillers, and private citizens. However, there is no real substitute for test well data. The ease of obtaining such data varies widely depending upon a number of factors including the relative abundance of groundwater resources in the local area, the attitudes and practices of local well drillers, the nature of the subsurface materials, and the depth to water-bearing strata. Where subsurface conditions are favorable, experienced local well drillers may be willing to guarantee to locate sufficient quantities of high-quality water. In this situation, test wells involve little risk. When conditions are unfavorable, the risk factor increases dramatically, but the need for test wells increases also. Although they can be expensive, pump tests are usually a good investment. In fact, it is not possible to complete the design of a well system until the wells are actually opened and tested. In some cases exploration costs can be mitigated by converting test wells to production wells.

(2) Water quality. A word of caution concerning the quality of groundwaters is in order. It has long been widely believed that groundwater, especially that taken from deep wells, is relatively free of contamination of anthropological origin when compared to surface water. However, such an assumption can no longer be safely made. A study of groundwater in New Jersey (Page 1982) revealed that groundwaters (from more than 1,000 wells) exhibited the same pattern of contamination as did surface waters (from over 600 sites), and that groundwater was at least as contaminated as surface water. The toxic contaminants investigated included 27 light hydrocarbons, 20 heavy chlorinated hydrocarbons, and 9 metals. The concentrations of the majority of the substances were either not significantly different or were greater in the groundwater samples when compared to the surface water samples. Much of this type of contamination was generated by landfill disposal practices. In 1977, the USEPA (1977b) estimated that as much as 220 million metric tons of industrial wastes end up in land disposal areas each year. While the health significance of long-term exposure to low levels of many contaminants has yet to be determined, it is obvious that groundwater is not necessarily “purer” than surface water. The problem of groundwater contamination is complicated by the possibility of long lag periods (even many years) between application of contaminants to the soil and their appearance in aquifers used for water supply. On the whole, it is prudent to expend resources as necessary to determine if a potential groundwater source is, or is likely to be, so contaminated that it is rendered unacceptable. This is one justification for test wells and detailed sanitary surveys.

(3) Construction. Excellent guidance relative to well construction is available in USEPA (1975). In addition, state and local health departments usually provide detailed information concerning well construction within their jurisdictions. Where applicable, wells should be constructed in accordance with Standard A100-66 of AWWA (1966). Several key elements to be considered in well design are presented below.

(a) Types. Wells may be dug, bored, driven, jetted, or drilled. While no single construction method is universally superior, deep wells (more than 30 m (100 ft) deep) are usually constructed by percussion or rotary drilling. Drilling may also be used for shallow wells, but is often not the most economical technique for that purpose. Properly constructed drilled wells are usually more dependable and less likely to be contaminated than other types (Salvato 1982). There are, however, exceptions. Well construction is highly specialized and local conditions are quite variable. Therefore, it is advantageous to obtain the services of someone who is knowledgeable, experienced, and fully familiar with well construction in the project area before writing specifications and contract documents for constructing wells.

(b) Location. As a rule, wells should be located on fairly high ground to ensure against contamination by surface water. The site chosen should be as far away as is practicable from known sources of pollution such as septic tanks, cesspools, privies, sewer lines, sanitary landfills, hazardous waste disposal sites, feedlots, or barnyards. It is not possible to say exactly what a safe distance is without detailed information on the site. A good rule of thumb is to maintain a minimum distance of 30 m (100 ft) between shallow wells (less than 15 m (50 ft) deep) and possible sources of contamination with an even greater distance in karst topography. In addition, such wells should always be located hydraulically upgradient of the source of contamination. Groundwater flow in shallow aquifers often parallels that of surface flow, but this should be verified before final site selection is made. Consult the appropriate regulations prior to selecting the final well location.

(c) Casings. Well casings serve to provide a stable, uniform opening from the surface to the aquifer by preventing collapse of the well wall. They also serve to prevent the entry of possibly contaminated water from other waterbearing strata or the surface. Sometimes the casing is placed as the well is being drilled depending on the method of well construction. Sometimes lightweight temporary casings are used and then replaced if the well proves satisfactory. In order to seal the well against possible contamination, it is common practice to grout the region between the outside of the casing and the well hole. The casing should be large enough to accommodate equipment that must be lowered into the well (e.g., submersible pump) and strong enough to resist the forces and stresses to which it is exposed during placement and operation. Leakproof joints between casing segments are important; thus, welded or threaded connections are usually used. All things considered, the requirements favor black steel casings. However, various plastics can be used for this purpose and may be of use, especially when severe corrosion of iron or steel would occur. Local well construction regulations may not permit plastic well casing. Ordinarily, casing sizes vary from a minimum of about 100 millimeters (mm) (4 in.) in diameter for wells with yields of less than 200 liters (50 gallons) per minute to 600 mm (24 in.) or more for wells with yields of around 8000 to 11 000 liters (2000 to 3000 gallons) per minute. TM 5-813-1 specifies that, except when water requirements are small, the minimum diameter of deep wells should be 200 mm (8 in.).

(d) Screens. When water is to be removed from unconsolidated geologic formations, it will be necessary to install a well screen. The ideal screen would be designed to allow water to pass without significant resistance, while at the same time prohibiting the entry of solid particles into the well and preventing collapse of the walls. A variety of designs are available from equipment manufacturers and suppliers. The size of the screen required for a given installation depends upon the type of screen selected, its hydraulic capacity, and the expected pumping rate as well as other factors. Screen selection should be influenced heavily by possible effects of corrosion and encrustation and the difficulty of cleaning and replacement. Screen selection should be performed by someone experienced in well design.

(e) Alignment. A drilled well should be reasonably straight and plumb. Of the two, straightness is usually the more important since it determines if a vertical turbine or submersible pump of a given size can be installed in the well. However, deviations from plumb may cause excessive wear or reduction in performance of some pumps. Most well codes specify allowable tolerances. Typical specifications suggest that a well should not vary from the vertical by more than one well diameter per 30 m (100 ft) of length and that a well should be straight enough to allow a 9-m- (30-ft-) long dummy having an outside diameter 13 mm (0.5 in.) less than the casing to move freely to the lowest anticipated pump location (Steel and McGhee 1979).

(f) Development. Development is a technical term for the process of removing “fines” (silt, fine-grained sand, etc.) from the vicinity of the well screen. The term is also applied to well construction in general. Development is almost always practiced when the aquifer being tapped is in an unconsolidated formation and is usually needed in other situations, such as a rock-wall well. The basic technique is to alternate the direction of flow across the screen and thus flush the fines away. Hydrojetting, bailing, overpumping, intermittent pumping, surging with a surge block or compressed air, and backwashing are all used for this purpose. Surging with a surge block or compressed air or hydrojetting is usually the preferred method in screened wells. The result is that the well screen is surrounded by a highly permeable layer of “clean,” well-graded material that allows free flow into the well; thus, the yield is increased. Such wells are sometimes referred to as “naturally gravel packed.” When suitable natural material is not present, it may be necessary to enlarge the diameter of the bottom of the well and introduce well-graded gravel. Wells constructed in this manner are called “gravel packed.” The development method should be chosen with care since it is possible to inadvertently clog the well. One practice that encourages good development procedures is to build into the contract a bonus for capacity in excess of some stated amount and a penalty for lesser capacity.

(g) Testing. Following development, a pumping test should be performed. The major purpose of this test is to determine the yield and drawdown characteristics of the well. If the data are taken carefully, it is also possible to learn a great deal about the hydraulic characteristics of the aquifer. This is especially true if observations at a nearby well in the same aquifer can be made simultaneously. However, a well pumping test serves primarily to test the completed well as a hydraulic structure and not the aquifer itself. Detailed procedures for conducting well pumping tests are readily available elsewhere (American Association for Vocational Instructional Materials 1973; AWWA 1966; Campbell and Lehr 1973; Folks 1977); and guidance is also available from the U.S. Geological Survey, state and local health departments, state geological survey agencies, well equipment manufacturers, etc., and is not presented herein. However, a few especially important points deserve mention. One is that the quality of information gleaned from a pumping test is closely linked with the accuracy to which determinations of flow rate and pumping depth are made. A second factor is that the temporary pump selected for the test should have a capacity at least 50 percent in excess of that of the pump planned for permanent installation. An even better approach is to select a pump having a capacity equal to or greater than the expected yield of the well. Thirdly, the discharge of the test pump should be easily controlled so that tests can be performed at several flow rates. The minimum flow rate needed is usually about 50 percent of the maximum. Deep well turbines are suitable for this purpose, as is any type of pump powered by a gasoline motor (can be throttled to vary flow rate). Finally, the pumping test should be continued long enough to provide a high degree of confidence in the results. There is no way to say in advance how long will be necessary, but 24 to 48 hours is a good estimate. When the well yield is not guaranteed by the driller, it is good practice to write the contract for this portion of the work on a per hour basis. This way it is not to the driller's advantage to end the test prematurely. When yield is guaranteed, the contract documents should clearly state the basis on which yield will be determined.

(h) Preventing contamination. Regardless of the construction method used, the well must be sealed effectively to prevent the entry of any water except from the aquifer being tapped. There are several techniques that may be used to accomplish this. One is to fill the region between the outside of the casing and the well wall with neat grout. The best method is to pump the grout in from the bottom up. Another technique is to extend the top of the casing at least 0.3 m (1 ft) above the pump house floor when applicable. The use of a pitless adapter (i.e., no well pit) and an effective sanitary seal is also important. Where practical, the area around the top of the well should be covered with concrete sloped to divert runoff away from the well. Other means of diversion may also be employed. It is usually better to drill a new well rather than try to extend a large-diameter well (e.g., a dug well) by drilling in the bottom. The old well will almost always serve as a source of contamination for the new well. Care should also be taken to avoid contamination, accidental or otherwise, during construction. Failure to exclude undesirable water is a frequent cause of well contamination. Wells generally should be vented unless the pump utilized demands an airtight installation (this is true for certain types of jet pumps). The vent should be considered as a possible source of contamination and located accordingly. It is good practice to extend the vent at least 600 mm (2 ft) above the highest known flood level, turn the opening downward, and cover it with a screen.

(i) Disinfection. Once construction is completed, the well should be cleaned of all ropes, oil, grease, timbers, pipe dope, tools, cement, etc., and disinfected. The standard procedure (AWWA 1966) calls for chlorine to be added to the well in a sufficient amount to produce an initial theoretical concentration of at least 50 mg/L and then remain in the well for at least 2 hours. A longer period, e.g., 24 hours, is better. Virtually any form of free available chlorine and any technique for application may be used. (To be effective, the chlorine must be in a valence state greater than -1. Therefore, chlorides are not useful.) Calcium hypochlorite is a popular dry form and sodium hypochlorite, a liquid, is readily available in several strengths as well as household bleach (about 50 000 mg/L free available chlorine). All equipment in contact with the water should also be disinfected (e.g., the pump). Following disinfection, the chlorine solution should be pumped out and the well should be sampled and tested for total coliform organisms. The absence of any coliforms is taken as evidence of disinfection. Samples should then be taken and subjected to chemical and physical analysis to ensure that the water is suitable for human consumption. The disinfection process should be repeated whenever the well is opened for maintenance (e.g., pump replacement), or if excessive coliform organisms are detected by routine testing.

(j) Number of wells. The number of wells required is a function of the total need for water, the yields of individual wells, the desired operating schedule, the water storage facilities available, regulatory requirements, and the desired excess capacity. It is advantageous to have at least two wells if economically feasible, and it is good practice to construct enough wells to meet average daily needs in substantially less than a full day of operation. TM 5-813-1 specifies an operating day of 16 hours (or less), and a minimum of two wells except for very small camps, or when flowing artesian wells or springs serve as the source. It should be noted that all wells should be provided with some way to measure water levels.

(k) Abandoned wells. On occasion, it is necessary that a well be closed, for example, a test well or an existing well that will no longer be used. Failure to properly seal such wells can lead to the contamination of entire aquifers. This has already occurred in some locations. AWWA (1966) presents a procedure to be used. Local regulatory agencies may have their own specifications.

5-5. Surface Water

a. General. Surface water is usually the source of last resort for small water systems because surface water almost invariably requires substantial treatment prior to use. Treatment, of course, requires a treatment plant, which, in turn, requires considerable capital investment and operation and maintenance effort. The picture is made even less favorable by the fact that surface water quality usually varies to such an extent that even the most automated treatment plants require considerable operator attention. Thus, the economics are often unfavorable. However, many times conditions are such that surface water is a viable alternative or the only feasible choice. The diversity of surface waters is so great that extended discussion herein would serve no useful purpose. However, some key design elements are considered briefly in the following sections.

b. Sanitary survey. The purposes and some major elements of the sanitary survey were presented in Section 5-2. Other specific points of interest for surface water supplies are listed below. The particular source being investigated will dictate the principal thrust of the study.

(1) Topography.

(2) Geology.

(3) Land use.

(4) Vegetative cover.

(5) Rainfall (amount and distribution).

(6) Streamflow and surface runoff patterns.

(7) Adequacy of the supply including seasonal effects.

(8) Wastewater discharges (type, location, strength, quantity, type of treatment provided).

(9) Necessity for an impoundment.

(10) Potential reservoir sites.

(11) Development costs.

(12) Legal constraints (use doctrine, prior rights).

(13) Historical water quality.

(14) Potential for protection of water quality.

(15) Future plans of other users.

c. Types. Surface water supplies are generally of one of three basic types: unregulated streams, impoundments, or natural lakes. Lengthy discussion of these categories is unwarranted since local conditions vary so widely. However, some general points of interest are presented in the following sections.

(1) Unregulated streams. Wide variations in both streamflow and water quality make unregulated streams a poor choice in most cases. If such a stream is chosen, the dry-weather flow should be estimated carefully since it determines the safe yield. If the maximum demand is greater than the safe yield, alternative sources, such as wells, must be developed or water shortages will occur. Even if the flow is sufficient, water quality may still be a serious problem, especially for small water systems, since close attention to treatment plant operation will be required. Off-stream raw water storage can help alleviate both problems, but can be rather expensive.

(2) Large lakes and impoundments. Large lakes or impoundments are often good sources of supply if they are located so that transmission costs are not excessive. The quality of such waters changes seasonally, but in a somewhat predictable fashion and, on a day-to-day basis, is less variable than for unregulated streams. The effects of varying water quality can be offset to some degree by the flexibility to take water from different depths. This is especially effective for deeper bodies of water that undergo a seasonal thermal stratification/destratification cycle. Lakes or impoundments receiving significant wastewater discharges should be viewed with caution since the buildup of nutrients such as phosphorous and nitrogen may lead to excessive algal productivity. Algae can cause operational difficulties for treatment processes, e.g., filtration, and can produce a wide variety of taste and odor problems. In addition, algae are producers of trihalomethane precursors; thus their presence can complicate disinfection or require additional treatment.

(3) Small lakes and impoundments. Small impoundments and natural lakes may be good sources of supply. This is especially true when the water system can own or control the entire watershed. This arrangement allows the water resource to be managed in such a way to protect and enhance water quality and ensure that competing uses do not adversely affect water supply. Economical yields of 75 to 90 percent of the annual streamflow can be realized in favorable situations. Methods for determining the storage volume needed to meet specific water demands were introduced in Section 4-3 and are given excellent coverage in many water supply oriented textbooks (Clark, Viessman, and Hammer 1977; Fair, Geyer, and Okun 1966a; Salvato 1982; Steel and McGhee 1979; and Viessman et al. 1977). The Federal Highway Administration (Folks 1977) has identified the following characteristics of the ideal small rural water supply watershed:

(a) Clean.

(b) Grassed.

(c) Free of contamination sources (barns, feedlots, privies, septic tanks, and disposal fields, etc.).

(d) Protected from erosion.

(e) Protected from drainage from livestock areas.

(f) Fenced to exclude livestock.

In addition, they suggest the following criteria for the impoundment or lake:

(a) At least 2-1/2 m (8 ft) deep at the deepest point.

(b) Maximum possible water storage in areas more than 1 m (3 ft) deep.

(c) Able to store at least a one-year water supply.

(d) Fenced.

(e) Free of weeds, algae, or floating debris.

d. Water quality and treatment. Water treatment facilities represent a significant portion of the total cost of a typical surface water system. The specific operations and processes required are determined by a combination of raw water quality, desired finished water quality, and regulatory requirements. Conventional surface water treatment involves removal of turbidity followed by disinfection. Processes used for these purposes are virtually required by regulation regardless of the quality of the raw water. Additional processes and operations for iron and manganese removal, softening, taste and odor control, etc., can be easily integrated into the treatment scheme, but contribute significantly to both first and continuing costs. Therefore, surface waters requiring specialized treatment should be avoided when possible. Sources exhibiting wide or rapid changes in water quality should also be avoided since such variations increase operating difficulty considerably. For more information that may be useful in choosing among alternative sources on the basis of the treatment required, the reader is referred to standard works on water treatment (AWWA 1971, 1990; Clark, Viessman, and Hammer 1977; Fair, Geyer, and Okun 1966a, 1966b; Salvato 1982; Sanks 1978; Steel and McGhee 1979).

e. Intakes. Intake systems are required to remove the water from the source and deliver it to transmission facilities. Design of intakes is highly site-specific; however, most can be categorized as submerged or exposed tower types. Regardless of the system chosen, intakes should be located well away from wastewater or stormwater discharges or other potential sources of contamination. Other factors that may impact on the design of intakes are type of source; water depth; bottom conditions; navigation requirements; effects of floods, currents, and storm or bottom conditions, and exposed structures and pipelines; prevalence of floating materials; and freezing.

(1) Submerged intakes. Submerged intakes may be applicable to lakes, streams, and impoundments, and are frequently utilized by small water systems. A common design consists of a wooden crib held in place by riprap or concrete. The inlet ports lead directly to submerged pipelines, and are covered by wooden slats that act as a screen. Inlet velocities are kept quite low so that clogging does not occur. This type of intake is located where bottom materials are stable and there is no interference with navigation. The pipelines carry the water to a pumping station located on shore. Such installations usually require very little maintenance. Another approach is to simply extend submerged pipes, with special fittings (e.g., flared end with strainer or a section of well screen) attached, into the water. The pipelines can be supported at the desired depth or held in place by a system of floats and anchors if flexible piping is used. Movable intakes have been used when the water depth varies over a large range, but they tend to be troublesome and require considerable attention.

(2) Tower intakes. More elaborate intakes consisting of exposed towers with multiple inlets are frequently used for larger flows. These systems can be very complex and may include automatically cleaned screens, pumping stations, and even living quarters.

(3) Infiltration galleries. When bottom conditions are unstable or water surface elevations fluctuate widely, infiltration galleries should be considered. These types of installation may be considered as surface or groundwater intakes since they perform essentially as horizontal wells, but are located at shallow depths and very near surface water sources. Typical designs call for well screens or perforated pipes to be laid near the edge of the water source, at an elevation below the lowest water level. Occasionally the infiltration gallery may be constructed directly under the water source rather than alongside it. Water flows through the soil from the surface source to the intakes, and is pumped out to a treatment facility. Water quality is similar to that typically expected from shallow wells. Frequently the only treatment required is disinfection. The use of this type of intake may result in substantially reduced treatment plant construction costs, as well as lower operation and maintenance requirements.

Chapter 6
Water Treatment

6-1. Introduction

The purpose of water treatment is to do whatever is necessary to render a raw water suitable for its intended use. Since both raw water qualities and intended uses vary, water treatment must be carefully tailored to fit individual situations. Even the most basic forms of water treatment require some operator time on a daily basis. More sophisticated plants may require almost constant attention. Thus, it behooves the designer of any water supply system, especially a small one, to give a great deal of attention to operational complexity when selecting water treatment techniques. From this viewpoint, the best treatment is no treatment at all. That is, the best approach is to locate a water source that requires no treatment. When the intended use is human consumption, regulatory requirements effectively limit such sources to existing water supply systems. Thus, any new source is probably going to require some treatment. When high-quality groundwater is available, it may be possible to limit treatment to disinfection. On the other hand, even the highest quality surface water will require turbidity removal in addition to disinfection. This is a major reason why groundwater is often preferred over surface water. Additional treatment will be required to deal with special problems such as tastes, odors, hardness, etc. Designers of small water systems should keep treatment facilities as simple as possible. Table 6-1 lists the BAT for drinking water contaminant removal. It is included only to indicate that removal of various constituents can be complex. However, the removal and the degree of removal are specified by state and local regulations and should be adhered to rather than this manual. As has already been pointed out in this manual, the degree of regulation for removal of contaminants can be affected by the quantity of water consumed.

a. Assumptions. In this chapter various operations and processes commonly used in water treatment are introduced and briefly discussed. Attention is deliberately focused on the typical needs of small water systems; thus, no attempt has been made to address every possible situation. The general approach is to consider typical treatment problems, briefly describe the most likely alternative means of treatment, alert the reader to the principal requirements and most important design considerations, and point out sources of more specific information, should such be desired. A fairly extensive list of references is included in Appendix A; however, no attempt has been made to present an exhaustive literature review. Many of the publications cited have rather complete lists of references on the particular subjects covered. Thus, the reader may be led to the desired information through a series of reference citations. Cookbook approaches to design are not offered. It is assumed that the intended use of the water is human consumption, that the raw water quality is known, and that raw water is available in sufficient quantities to allow for bench and pilot scale treatability studies. Chapter 3 discusses water quality requirements, common constituents of water, and the regulatory framework for protection of the safety and integrity of public water supplies. Emphasis is given to treatment required of typical ground and surface waters. However, many of the references cited also deal with more exotic problems. The USEPA "Manual of Treatment Techniques for Meeting the Interim Primary Drinking Water Regulations" (1977a) is a good general reference on water treatment.

b. Nomenclature. Many water treatment professionals make a distinction between "operations" and "processes." Because this distinction is somewhat arbitrary and is not rigorously adhered to, some confusion is inevitable. As a rule, "operations" involve the application of physical principles and forces, while "processes" involve chemical reactions or biological activity. Therefore, screening, straining, and settling technically are operations, while chlorination and chemical coagulation are processes. Sometimes no clear distinction is readily apparent. Examples include adsorption of organic substances onto activated carbon and granular media filtration. Thus, some authors use the terms virtually interchangeably. This approach is used in this manual.

c. Design basis. Water treatment is an old, highly specialized, and largely empirical technical field that is strongly influenced by considerable conservatism with respect to protection of public health and the monetary investments of the public in water supply systems. Therefore, the field has traditionally been slow to accept new technology. However, increased research activity, motivated largely by public interest in the linkage between environmental factors and the quality of life (especially health), serves to make it quite likely that water treatment practices unknown today will be commonplace in the near future. In the meantime, the design of water treatment facilities is heavily influenced (dictated in many cases) by regulatory requirements based on previously successful practice. In these circumstances the job of the designer is often to find the most economical design that satisfies the regulatory agency with jurisdiction. However, many water treatment techniques can be modeled in the laboratory, or on a pilot scale, with relative ease. It is recommended that, where appropriate, such tests be perfomed and that the results, along with regulatory guidelines, serve as the basis for design. Information relative to bench and pilot scale studies is widely available from sources including Clark, Viessman, and Hammer (1977), Fair, Geyer, and Okun (1966b), Hudson (1981), Sanks (1978), and Weber (1972). Many engineers understand water treatment fairly well from a mechanistic or operational point of view, but have little appreciation for the

Table 6-1
Best Available Technologies for Drinking Water Contaminant (adapted from AWWA)

Contaminant	Conventional Processes	Specialized Processes
	Microbials	
Cryptosporidium	C-F SSF DF DEF D	
E. coli	D	
Fecal coliforms	D	
Giardia lamblia	C-F SSF DF DEF D	
Heterotrophic bacteria	C-F SSF DF DEF D	
Legionella	C-F SSF DF DEF D	
Total coliforms	D	
Turbidity	C-F SSF DF DEF D	
Viruses	C-F SSF DF DEF D	
	Inorganics	
Antimony	C-F[1]	RO
Arsenic	NA	
Asbestos (fibers/1>10 µm)	C-F[1] DF DEF CC	IX RO
Barium	LS[1]	IX RO
Beryllium	C-F[1] LS[1]	AA IX RO
Bromate	DC	
Cadmium	C-F[1] LS[1]	IX RO
Chlorite	DC	
Chromium (total)	C-F LS[1] (Cr III)[1]	IX RO
Copper	CC SWT	
Cyanide	CL2	IX RO
Fluoride		AA RO
Lead	CC SWT PE LSLR	
Mercury	C-F[2]+[1] LS[1]	RO[2] GAC
Nickel	LS[1]	IX RO
Nitrate (as N)		IX RO ED
Nitrite (as N)		IX RO

(Sheet 1 of 4)

Note: Abbreviations used in this table: AA - activated alumina; AD - alternative disinfectants; AR - aeration; AX - anion exchange; CC - corrosion control; C-F - coagulation-filtration; CL2 - chlorination; D - disinfection; DC - disinfection-system-control; DEF - diatomaceous earth filtration; DF - direct filtration; EC-enhanced coagulation; ED - electrodialysis; GAC - granular activated carbon; IX - ion exchange; LS - lime softening; LSLR - lead service line removal; NA - not applicable; OX - oxidation; PAP - polymer addition practices; PE - public education; PR - precursor removal; PTA - packed-tower aeration; RO - reverse osmosis; SPC - stop prechlorination; SWT - source water treatment; SSF - slow sand filtration.
[1] Coagulation-filtration and lime softening are not BAT for small systems.
[2] Influent <= 10 µg/L.
[3] Sum of the concentrations of bromodichloromethane, dibromochloromethane, tribromomethane, and trichloromethane.

Table 6-1. (Continued)

Contaminant	Conventional Processes	Specialized Processes
	Inorganics (continued)	
Nitrite & Nitrate (as N)		IX RO
Selenium	C-F(Se IV)[1] LS[1]	AA RO ED
Sulfate		IX RO ED
Thallium		AA IX
	Organics	
Acrylamide	PAP	
Alachlor		GAC
Aldicarb		GAC
Aldicarb sulfone		GAC
Aldicarb sulfoxide		GAC
Atrazine		GAC
Benzene		GAC PTA
Benzo(a)pyrene		GAC
Bromodichloromethane		EC
Bromoform		EC
Carbofuran		GAC
Carbon tetrachloride		GAC PTA
Chloral hydrate		EC
Chlordane		GAC
Chloroform		EC
2,4-D		GAC
Dalapon		GAC
Di(2-ethylhexyl) adipate		GAC PTA
Di(2-ethylhexy) phthalate		GAC
Dibromochloromethane		EC
Dibromochloropropane (DBCP)		GAC PTA
Dichloroacetic acid		EC
p-Dichlorobenzene		GAC PTA
o-Dichlorobenzene		GAC PTA
1,2-Dichloroethane		GAC PTA
1,1-Dichloroethylene		GAC PTA
cis-1,2-Dichloroethylene		GAC PTA

(Sheet 2 of 4)

Table 6-1. (Continued)

Contaminant	Conventional Processes	Specialized Processes
	Organics (continued)	
trans-1,2-Dichloroethylene		GAC PTA
Dichloromethane (methylene chloride)		PTA
1,2-Dichloropropane		GAC PTA
Dinoseb		GAC
Diquat		GAC
Endothall		GAC
Endrin		GAC
Epichlorohydrin	PAP	
Ethylbenzene		GAC PTA
Ethylene dibromide (EDB)		GAC PTA
Glyphosate		OX
Haloacetic acids[1]		
(Sum of 5; HAA5)		EC
-		EC+GAC
Heptachlor		GAC
Heptachlor epoxide		GAC
Hexachlorobenzene		GAC
Hexachlorocyclopentadiene		GAC PTA
Lindane		GAC
Methoxychlor		GAC
Monochlorobenzene		GAC PTA
Oxamyl (vydate)		GAC
Pentachlorophenol		GAC
Picloram		GAC
Polychlorinated byphenyls (PCBs)		GAC
Simazine		GAC
Styrene		GAC PTA
2,3,7,8-TCDD (dioxin)		GAC
Tetrachloroethylene		GAC PTA
Toluene		GAC PTA
Toxaphene		GAC

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Table 6-1. (Concluded)

Contaminant	Conventional Processes	Specialized Processes
	Organics (continued)	
2,4,5-TP (silvex)		GAC
Trichloroacetic acid		EC
1,2,4-Trichlorobenzene		GAC PTA
1,1,1-Trichloroethane		GAC PTA
1,1,2-Trichloroethane		GAC PTA
Trichloroethylene		GAC PTA
Total Trihalomethanes[3]	AD PR SPC	
(sum of 4)		
-		EC
-		EC+GAC
Vinyl chloride		PTA
Xylenes (total)		GAC PTA
	Radionuclides	
Beta-particle and photon emitters	C-F	IX RO
Alpha emitters		
-	C-F	RO
Radium 226+228		
Radium 226	LS[1]	IX RO
Radium 228	LS[1]	IX RO
Radon	AR	
Uranium	C-F[1] LS LS[1]	AX

(Sheet 4 of 4)

chemistry that makes many of the processes work. A deeper understanding of fundamental process chemistry can lead to designs that are more economical and effective than those developed solely by application of traditional criteria. Benefield, Judkins, and Weand (1982) is especially useful in this regard. General information on design of water treatment facilities is plentiful (AWWA 1971, 1990; Clark, Viessman, and Hammer 1977; Culp and Culp 1974; Fair, Geyer, and Okun 1966b; Hamann and Suhr 1982; Hammer 1975; Hudson 1981; Merritt 1976; Nalco Chemical Company 1979; Sanks 1978; Steel and McGhee 1979; and Weber 1972). Some specific guidance for Corps of Engineers projects is presented in TM 5-813-3. Regulatory agency personnel and publications are excellent sources of information pertinent to specific problems and local requirements. Much can be learned by observing existing water treatment plants, looking over operating records, and discussing operational problems with knowledgeable plant personnel. Other major information sources include the AWWA Standards, EP 310-1-5, and equipment manufacturers and suppliers. Problems specific to small water systems are discussed in Clark (1980), Clark and Morand (1981), Hansen, Gumerman, and Culp (1979), Lehr et al. (1980), Morand et al. (1980), and Stevie and Clark (1980, 1982).

6-2. Disinfection

Disinfection involves the removal, destruction, or inactivation of pathogenic (disease-causing) organisms, and will be discussed first since it is often the only form of treatment required for small water systems. The effectiveness of disinfection is generally determined indirectly via enumeration of coliform organisms in the treated water (paragraph 3-4*d* and 3-7). Ideally none should be present. The USEPA proposed regulation for disinfectants and disinfection by-products in 1994. It is anticipated that the practice will continue to change, especially in response to maximum allowable levels of suspected health-risk by-products. Changes in operations and treatment will continue as new developments in technology, scientific knowledge, and regulations occur.

a. Alternative methods. Disinfection may be accomplished by a number of means, including the application of

(1) Heat (e.g., pasteurization).

(2) Radiation (e.g., ultraviolet light).

(3) Heavy metals (e.g., silver).

(4) Oxidizing chemicals (e.g., chlorine, iodine, hydrogen peroxide, ozone).

Each method has its uses, but economics and public health considerations favor the use of oxidizing chemicals for potable water treatment. Within this category, chlorine has been and probably will continue for many years to be the disinfectant of choice for most water supply systems, especially the smaller ones (AWWA 1982; Hoff and Geldreich 1981; and Rice et al. 1981). When disinfectants are applied to surface waters and groundwaters under the influence of surface waters, the product of *C* and *T*, i.e., residual disinfectant concentration times the disinfectant contact time, must meet minimum requirements as specified in tables provided by the USEPA. In addition, consideration must be given to the development of disinfectant by-products. State regulatory agencies should be contacted for current regulatory levels for any and all by-products. The resulting concentrations may have an impact on the disinfectant selected.

b. Chlorination.

(1) Advantages and disadvantages. Some advantages of disinfection with chlorine are listed below:

(a) Relatively low cost.

(b) Ease of application.

(c) Proven reliability.

(d) Easy detectability.

(e) Residual disinfecting power.

(f) Familiarity with its use.

(g) Used for other treatment purposes (e.g., oxidation).

The principal disadvantages of chlorination are that in some cases undesirable tastes and odors may be produced (e.g., reactions with phenols) and chlorination of some organic substances produces compounds known or suspected to be hazardous to human health. One group of such compounds, the trihalomethanes or THMs (e.g., chloroform), are already the subject of Federal regulation (para 3-4*d*(4), Kavanaugh et al. 1980; Krabill 1981; Singer et al. 1981; and Vogt and Regli 1981).

(2) Chemistry of chlorine. When chlorine is added to water, a variety of reactions may take place, but not all at the same rate. The difference between the amount of chlorine added to a water (the dosage) and the amount remaining at any given time (the residual) is a measure of the amount that has reacted and is referred to as the "demand." The chlorine residual is, therefore, a measure (though not a perfect one) of the potential for continuing disinfection. The products of reactions between chlorine and ammonia (NH_3) or the ammonium ion (NH_4^+) are of special interest because they possess some disinfecting ability of their own. Collectively these products are called "chloramines" and are often referred to as "combined available chlorine." It is necessary to distinguish the combined available chlorine residual from the "free available chlorine residual" (i.e., chlorine that has reacted only with the water itself), because the effectiveness of the latter is much greater than that of the former in the pH range (6 to 9) of interest in water treatment. Chlorine chemistry is covered very well in most water supply textbooks (e.g., Clark, Viessman, and Hammer 1977; Fair, Geyer, and Okun 1966b; Hammer 1975; Sanks 1978; and Steel and McGhee 1979).

(3) Required residual. Regulatory requirements vary considerably, but adequate disinfection of relatively clear water (turbidity < 5 NTU) can usually be accomplished by maintaining a free available chlorine residual of 1 mg/L for at least 30 minutes at a pH < 8. Higher residuals and/or longer contact periods will provide an increased level of protection from pathogens. Reductions in contact time and/or high pHs should be compensated for with higher residuals. A residual of 0.2 mg/L free available chlorine throughout the distribution system will minimize risks associated with possible recontamination of treated water; however, state regulations call for minimum and perhaps maximum values at various points within the water treatment system.

(4) Alternative forms of chlorine. Chlorine is available in several forms. A brief discussion of those commonly used in water treatment is presented below.

(a) Chlorine gas. Liquified chlorine gas is by far the most popular form of chlorine for use at larger water treatment plants. It is relatively inexpensive, especially when purchase in railroad tank car lots is feasible. However, chlorine gas is extremely hazardous and special precautions such as a separate chlorination room or facility, floor level ventilation, and provision of safety equipment such as gas masks must be undertaken when it is used. As a result, gas chlorination is often unsuitable for small water systems, although several equipment suppliers have small gas chlorinators available in their product lines.

(b) Calcium hypochlorite. Calcium hypochlorite is a dry powder or granular material that is widely used for small installations such as home swimming pools. The commercial form has a long shelf life and is safer to handle than chlorine gas (although all forms of chlorine used for disinfection are hazardous to some degree), but contains a significant insoluble, inert fraction. Typical products are 60 to 65 percent available chlorine by weight. When calcium hypochlorite is used to disinfect water, the dry form is mixed with water and the insoluble fraction is allowed to settle. The liquid is then drawn off and used as a stock solution to disinfect the water supply. Failure to separate the liquid from the insoluble residue may result in clogging or otherwise damaging equipment. For small systems, this process may be a disadvantage since significant operator time is required. The other major disadvantage is cost. On an equivalent basis calcium hypochlorite is up to six times as expensive as chlorine gas in small 150-pound cylinders.

(c) Sodium hypochlorite. Sodium hypochlorite is probably the best form of chlorine for small water systems. It is commercially available as a clear liquid containing between 12 and 17 percent available chlorine and is marketed in containers as small as 2 L (1/2 gal). Some very small water systems can use household bleaches such as Clorox or Purex, which are dilute (about 5 percent available chlorine) solutions of sodium hypochlorite. The major advantages of sodium hypochlorite are that it is relatively safe to use, and since it is already a liquid, little handling or processing is required prior to use. Costs are similar to those of calcium hypochlorite. The major disadvantages of sodium hypochlorite are that it has a half-life of approximately 90 days so it cannot be kept for long periods and it presents a chlorine gas danger if mixed with acid or ferric chloride.

(5) Hypochlorinators. Several types of hypochlorite solution feeders, called hypochlorinators, are available. The best type for small water systems is the positive displacement pump variety. These devices make use of a small metering pump that can be precisely adjusted to deliver the hypochlorite solution at a given rate against a wide range of resisting pressures. Typically, operation of the metering pump is synchronized with that of the water pump so that when water is flowing, the chlorine solution is automatically fed into either the suction or discharge piping in the proper proportion. Duplicate units should be provided so that disinfection will not be interrupted. Most positive displacement hypochlorinators are electrically operated, but water-powered models are also available. It is good practice to provide some type of sensing device on the chlorine solution tank that will set off an alarm or automatically shut down the water pumps when the solution level drops too low. Suction and aspirator feeders are also available to feed chlorine solutions, and tablet chlorinators that use pelletized calcium hypochlorite are marketed by several firms. Tablet, aspirator, and suction chlorinators are usually more difficult to control and less dependable than the positive displacement type. Chemical compatibility must be evaluated for all components such as pump and pump parts, solution tanks, and piping/tubing. The specifications and drawings should require that the manufacturer certify his equipment for the proposed service.

(6) Chlorine dosage. The proper chlorine dosage depends upon a number of factors including the

(a) Chlorine demand.

(b) Contact period.

(c) Residual.

(d) Temperature.

(e) pH.

Unfortunately there is no way to determine the required dosage directly without experimentation. However, under normal conditions, no more than 2 or 3 mg/L will be required. Higher demands, shorter contact periods, lower temperatures, and/or pH above about 8 will increase the required dose. Compliance with the disinfection rule must be achieved as required for various treatment techniques as determined by state environmental authorities.

(7) pH control. pH is a very important factor in the control of chlorination. When sufficient chlorine (gas or hypochlorite) is added to a water to produce a free available residual, a chemical equilibrium is established between the hypochlorous acid molecule (HOCl) and the hypochlorite ion (OCl^-). This equilibrium is controlled by pH. The two forms of free available chlorine are present in roughly equal amounts at a pH around 7.7. At a lower pH the acid predominates, and at a higher pH the ion is more prevalent. The third possible form of free available chlorine, the Cl_2, does not exist in

solution at pH levels high enough to be of interest in water treatment. (Note: if pH drops to as low as 2 or 3, gas may be evolved. This is an extremely hazardous situation.) The equilibrium between acid and ion is established regardless of the form in which the chlorine is added (gas or hypochlorite); however, the net effect on pH is not the same. Addition of the gas will destroy alkalinity and lower the pH, while addition of hypochlorite will tend to raise the pH. It is important to hold the pH in the 5 to 8 range if possible, since the acid molecule is a far more effective disinfectant than the ion. Thus, it may be necessary to add a chemical such as sodium bicarbonate or sulfuric acid along with chlorine to adjust the pH to the desired range.

(8) Superchlorination-dechlorination. For some small water systems, it is difficult or impossible to ensure an adequate contact time for ordinary chlorination. In these cases it is possible to superchlorinate, that is, to add more chlorine than would ordinarily be necessary, and then remove the excess (dechlorinate) prior to use. Dechlorination can be accomplished chemically by addition of a reducing agent such as sodium sulfite, sodium bisulfite, or sodium thiosulfate, or by activated carbon adsorption. However, chemical methods are difficult to control precisely enough to leave a consistent residual, and activated carbon adsorption can be expensive. Where possible it is probably better to provide additional contact time (e.g., by making the pressure tank on hydropneumatic systems bigger or by storage reservoir addition and using a repumping operation) than to attempt superchlorination-dechlorination.

(9) Chlorine-ammonia treatment. When chlorination produces undesirable tastes and odors, or when the production of chlorinated organic compounds must be minimized, chlorine-ammonia treatment may be used. The controlled addition of both substances together results in a combined available chlorine (chloramine) residual that does not react with phenols to produce taste and odor problems and does not produce chloroform or similar compounds. Chloramines are much less effective as disinfectants than either hypochlorous acid or hypochlorite, but they are very persistent and can provide some level of protection for an extended period. The cost and operational complexity of this technique should be evaluated versus other measures such as carbon adsorption for precursor removal.

(10) Chlorine dioxide. Chlorine dioxide ClO_2, is a powerful oxidant that has excellent germicidal properties, is unaffected by pH in the range normally encountered in water treatment, and does not react with ammonia. It has been used successfully for control of tastes and odors, especially those produced by phenols, but is seldom used in the United States for disinfection. Chlorine dioxide does not react with water; thus its chemistry is quite different from that of the more commonly used forms of chlorine. Its principal advantage for potable water disinfection is that it apparently can oxidize organic substances without producing halogenated hydrocarbons, such as chloroform. However, it is a very dangerous gas that must be produced onsite, and the health effects of possible by-products of its use are unknown or poorly defined. Thus, for the present, chlorine dioxide is unlikely to be the disinfectant of choice for small water systems, although low-capacity generation systems are available. An excellent discussion of chlorine dioxide chemistry, generation, use, etc., is presented by White (1978).

c. Iodination. Iodine is an excellent disinfectant, but is much more expensive than chlorine (as much as 20 times) and has possibly deleterious health effects, especially for unborn children and individuals with thyroid problems. While the extent of these effects is not fully known, it seems reasonable that iodine can be safely used as a disinfectant for public water supplies serving only a transient population, or in short-term (up to 3 weeks) emergency situations. The combination of unfavorable economies and possible health effects makes continuous use of iodine unwarranted (Folks 1977; Weber 1972).

d. Ozonation. Ozonation for disinfection of public water supplies has been practiced in Europe for many years and is gaining popularity in the United States. Ozone, O_3, does not form trihalomethanes or other substances presently known to have deleterious health effects. Thus, the process is attractive for large systems where a potential trihalomethane problem exists. There are three principal disadvantages of ozonation for disinfection of small water supplies. One is that ozone is so unstable that no residual can be maintained. This can be overcome by using ozone as the primary disinfectant and maintaining a residual with chlorine-ammonia treatment, for example. This process is far too complicated for most small water systems. A second problem is that ozone must be generated onsite. This complicates operation and maintenance problems and is energy intensive as well. Thirdly, ozonation is simply too expensive for many small systems. Ozone can be a safety hazard, and appropriate safety requirements must always be developed for each site.

e. Ultraviolet radiation. Ultraviolet (UV) radiation has been recognized for many years as having germicidal properties, and has been proposed for disinfection of water supplies since the early twentieth century. Commercially available UV radiation devices intended for water treatment are composed of a quartz sleeve housing one or more low-pressure mercury vapor lamps that radiate at a wavelength in the 250- to 260-nm range. The lamps themselves are similar to fluorescent bulbs without the coating to convert UV radiation to longer wavelength visible light. UV radiation disinfects by destroying the cell, or interfering with normal growth and development. In order to be effective, the radiation must be incident on each organism. Thus, suspended particles (e.g., turbidity) can shade and protect organisms. Substances such as iron compounds, phenols, other aromatic organic compounds, etc., are effective UV absorbers and can also reduce efficiency. The process can be designed to work automatically, requires minimal contact time, and produces no known undesirable by-products, and overdosing is not possible. However, the penetrating power of the radiation is low, the lamps slowly lose effectiveness, no residual disinfecting power is produced, there is no rapid test of effectiveness, efficiency is limited by the factors noted above, the equipment is expensive, and electrical power consumption is high. Presently, the use of UV radiation for disinfecting public water supplies is limited to very small systems, processing low-turbidity waters, having low concentrations of absorbing substances, when residual disinfecting ability is unimportant.

6-3. Iron Removal

Problems commonly associated with iron in water and possible removal techniques are introduced in paragraph 3-8*g*. The specific problem observed is related to the valence state of the iron. Ferrous iron Fe^{+2} is soluble in water and can cause taste problems. Ferric iron Fe^{+3} is much less soluble and tends to form precipitates that vary in color from yellow to brown to red. These particles make the water unsightly and can cause staining of plumbing fixtures, interfere with cleaning and washing activities, and impart an unpleasant taste to the water. In most natural waters, the ferrous form is readily oxidized to the ferric form by contact with molecular oxygen. Certain attached filamentous bacteria (*Crenothrix* and *Leptothrix*) derive energy by oxidizing iron and storing the oxidized form in their structure. These organisms are especially troublesome to water systems because they take up residence in piping systems. Occasionally clumps of organisms, in the form of gelatinous masses, break away and are entrained in the flow. Thus, periodic severe iron problems may result. The diversity of iron problems is such that control techniques must be tailored to fit specific situations. Therefore, an important first step is to determine what form the iron is in and what range of concentration can be expected. Secondly, the possibility that the iron is present as a result of corrosion should be investigated if applicable. It is generally better to control the corrosion process than remove the iron after corrosion occurs.

a. Polyphosphates. When the iron is in the ferrous (dissolved, colorless) state and the concentration is no more than about 3 mg/L, the most convenient approach may be to simply mask the problem by adding polyphosphates such as sodium hexamethaphosphate. These compounds act as chelating agents to sequester the iron prior to precipitation. Therefore, they should be applied before oxidation occurs. The sequestering process may be thought of simply as preventing the iron from entering into the reactions that produce precipitates. The iron is not removed from the water. Polyphosphate doses of 1 mg/L to 5 mg/L per mg/L iron present are typical. The process is simple and requires only a polyphosphate solution tank and feed systems similar to that used for hypochlorination systems (paragraph 6-2*b*(5)). As a bonus, polyphosphates may help control corrosion. However, hexamethaphosphates can begin to break down within 24 hours or less, reverting to an orthophosphate which has no sequestering capability. Also, orthophosphates can serve as a nutrient to water system bacteria and other microbes. Hence, retention time within the water system is a critical element for deciding on its use.

b. Ion exchange. Small amounts of ferrous iron may be removed by ion exchange type water softeners (paragraph 6-5*b*). However, ferric iron must not be present or severe fouling of the exchange media can occur. When this technique is used, manufacturers' recommendations with respect to the maximum allowable iron concentration and installation and operation of the exchanger should be followed rigorously. Maximum limits may range upward to 5 or 10 mg/L. However, experience in some Corps Districts has been that levels greater than 1 mg/L can cause resin fouling if iron bacteria contamination is present.

c. Oxidation-filtration. The most popular methods of iron removal involve oxidation of the iron by aeration, chlorination, or treatment with potassium permanganate, followed by some form of filtration. The rate of oxidation via aeration or chlorination is faster at higher pH. Thus, it may be necessary to adjust the pH to 7 or above to achieve satisfactory results. In larger plants a sedimentation step is often used to remove the settleable fraction of the precipitate and take some load off the filters. The filters used may be the gravity type, such as slow or rapid sand filters, or may be pressure operated. Pressure filters are commonly used for small systems, especially when double pumping can be avoided. The so-called oxidizing filter is a pressure type that is frequently employed. The media in oxidizing filters usually consists, at least in part, of natural greensand zeolite coated with oxides of manganese. These oxides promote rapid oxidation by serving as catalysts, or by actually entering into reactions, and may also serve as adsorption sites for ferrous ions. Frequently, a separate layer of granular material, such as sand or anthracite coal, is provided to act as a roughing filter to remove precipitates and prevent clogging of the oxidizing layer. This type of filter requires occasional backwashing. The zeolite and coating can be reactivated by occasional regeneration with potassium permanganate. Typical flow rates are up to 20 L per minute per square meter ($L/min/m^2$) (5 gallons per minute per square foot ($gal/min/ft^2$)) of nominal filter surface area. A sustained flow of at least 30 to 40 $L/min/m^2$ (8 to 10 $gal/min/ft^2$) of filter area must be available for a duration of several minutes to provide adequate filter bed expansion during backwashing. Volume expansions up to 40 percent may be required for cleaning. All types of filters, but especially pressure filters, perform better in continuous operation than in situations where frequent start/run/stop cycles are required. Thus, it may be beneficial, from the standpoint of filter effluent quality, to provide enough treated water storage capacity to minimize the number of on/off cycles required per day. Typically, if alkalinity is less than 100 mg/L as $CaCO_3$, manganese zeolite process for iron removal is not recommended without increasing the alkalinity.

(1) Aeration. Oxidation via aeration is frequently used for surface waters. The air may be applied through small bubble diffusers by a low-pressure blower, by spraying the water into the air, or by allowing the water to trickle down over a multiple-tray aerator. A typical design calls for three or four trays covered with coke or some other medium having a large surface area. The water is sprayed onto the top tray and allowed to trickle down over the lower trays. The high-surface-area medium serves to increase the area of the air/water interface and thus promote oxidation. The process also removes undesirable gases such as carbon dioxide and hydrogen sulfide. If the pH is maintained above about 7, the process can be very effective, especially when followed by filtration. An advantage of aeration is that the system cannot be overdosed; thus, precise operator control is not required. Devices for introducing oxygen under pressure are available for use with pressurized groundwater systems. However, it may not be desirable to oxygenate these waters since it will promote corrosion.

(2) Chlorination. Oxidation of iron with chlorine is effected by a number of variables including pH, chlorine dose, reaction time, mixing conditions, etc. Generally the process proceeds much slower than oxidation by permanganate. In some cases it may be necessary to superchlorinate and then reduce the residual following filtration. When this is required, granular activated carbon filters may be used to remove both the particulates and the excess chlorine. This can be quite expensive since the carbon will require occasional regeneration. For small plants, it may be less expensive to replace the carbon rather than regenerate onsite. The chlorine may be fed by solution feeders (paragraph 6-2*b*(5)). Superchlorination-dechlorination is discussed in paragraph 6-2*b*(8).

(3) Potassium permanganate. Ferrous iron is readily oxidized to the ferric form by potassium permanganate. The permanganate can be added to the water by solution feeders similar to hypochlorinators (paragraph 6-2*b*(5)). In theory, a potassium permanganate dose of 1 mg/L will oxidize 1.06 mg/L of iron; however, in practice it may be possible to use less than the theoretical dose. The reaction with permanganate is many times faster than that with chlorine, and is easily controlled since a small excess of permanganate produces a slight pink color. An additional advantage of this method is that pH within the range of about 5 to 9 does not significantly affect the reaction rate. The combination of permanganate addition, a short reaction time (contact tank), and an oxidation (greensand) filter is referred to as a potassium permanganate, continuous regeneration operation. If extra permanganate is added, it regenerates the greensand media. The process is most effective when the iron content of the water is fairly constant, but can work well if properly operated when the iron concentration varies up to 1.5 and 2.5 mg/L. In principle, where excess ferrous iron exceeds the $KMnO_4$ injection level, the greensand oxidizes the iron. Hence good operator control should require the checking for occasional pink downstream of the filter to assure complete recharge.

d. Iron bacteria. Iron bacteria problems can be effectively controlled by a rigorous chlorination program in concert with one of the other techniques discussed above. When iron concentrations are very low, continuous removal may not be needed and periodic superchlorination may be all that is required. One must always ensure that components transferred from one water system to another have been adequately disinfected prior to reinstallation to prevent the potential spread of iron bacteria and other organisms.

e. Summary. Iron removal problems can be frustrating since processes that work to control one form may be relatively ineffective against other forms. Thus, the importance of correctly identifying the true nature of the problem cannot be overemphasized. Pilot or bench scale studies can be very helpful in selecting a dependable process. For small water systems, dependability and ease of operation are key factors in design.

6-4. Manganese Removal

Manganese is much less common than iron, but is removed by essentially the same processes. Problems associated with manganese are discussed in paragraph 3-8*h*. Removal of manganese is complicated to a degree since oxidation proceeds most rapidly at a pH of around 9.5, especially when aeration or chlorination is used. Treatment with 2 mg/L or less potassium permanganate per mg/L of manganese is effective for oxidation to the insoluble form. Permanganate oxidation followed by an oxidation filter is a dependable removal process. Many times, iron and manganese problems are found together. Unlike the continuous regeneration operation, where the main objective is to remove iron, if the water contains only manganese or manganese with small amounts of iron to be removed, intermittent regeneration is recommended. Intermittent regeneration uses a $KMnO_4$ solution to fill, hold, and recharge the greensand filter after a specified quantity of water has been processed.

6-5. Hardness Removal

Hardness, which is usually composed almost entirely of calcium and magnesium, is introduced in paragraph 3-8f. The two basic methods of hardness removal (softening) are chemical precipitation and ion exchange. The former is widely used at larger plants, but is so operationally complex and expensive that it is almost never used for small installations.

a. Purpose. Hardness, at levels normally encountered, is not considered a health problem; therefore, removal is not mandatory. Thus consideration must be given to whether there is any real need to remove it. Such a question calls for the careful balancing of a combination of aesthetic and economic factors. Most people who are not used to hard water would probably prefer that water with a total hardness of more than about 125 mg/L as calcium carbonate be softened prior to use. However, unless public acceptance is a problem, it is difficult to justify softening a water containing a total hardness less than about 200 mg/L as calcium carbonate. Above about 300 mg/L as calcium carbonate, some removal is almost always necessary to protect equipment and piping systems.

b. Ion exchange softening. From an operational standpoint, ion exchange softening is a very simple process. A typical softener looks very much like a pressure filter in that it is a cylindrical container enclosing a packed bed of granular material. This material is called the exchange medium, and in modern softeners is usually a synthetic organic substance, such as polystyrene, formed in small beads. Typical bead diameters range from about 0.3 to 1.2 mm with the vast majority (95 percent or so) falling in the 0.4- to 0.8-mm region. Other types of exchange media are also marketed, and the nomenclature can be confusing since descriptive terms are used rather loosely. Originally, naturally occurring zeolite was the only choice. When technology to manufacture synthetic media came available, the term synthetic zeolite was used to describe some of the products. The term resin is also frequently used in reference to some types of media.

(1) Operation. Regardless of the specific medium used, it is a material that has many chemically active surface sites to which metallic ionic species are attracted. For freshly prepared (or regenerated) media these sites are predominantly occupied by monovalent species such as hydrogen or, more commonly in the case of potable water treatment, sodium.

(a) Removal. The water to be treated is passed through the softener at rates ranging from about 8 to 20 L/min/m^2 (2 to 5 gal/min/ft^2) of nominal exchanger surface area. As the water moves through the bed of medium, divalent ions in the water such as calcium and magnesium replace the monovalent species (sodium) in the medium because of their stronger affinity for the medium sites. Thus, divalent ions are "exchanged" for monovalent ions that do not contribute to hardness. There is no change in the electrochemical balance since one divalent ion replaces two monovalent ones. The process continues until the medium is "exhausted." At this point, few surface sites are available and divalent ions begin to break through into the treated water effluent. When this occurs, the medium must be regenerated. Ion exchange softeners are not intended to act as filters for suspended solids, although the beads are small enough to trap such material. Thus, it is important to pretreat turbid waters (turbidity 5 NTU) prior to softening. The media are not selective for calcium and magnesium; therefore, ferrous iron can be removed. However, most manufacturers recommend maximum concentrations of iron that should not be exceeded in the feed water, since excessive iron will tend to foul the media, reduce efficiency, and increase head loss through the bed.

(b) Regeneration. Regeneration is accomplished by passing a strong solution of the appropriate monovalent ion through the exchanger for a short period, or by "soaking" the medium in such a solution. For sodium type exchangers (the most common), a 10 to 15 percent solution of sodium chloride is used. Although less expensive grades are available, a purified salt should be used for regeneration. The next best choice would be a very clean rock salt. Typically rock salt contains considerable trash and has a significant insoluble fraction that will tend to clog the exchanger. An acid solution would be used for a hydrogen type medium. The exchange process reverses because of overwhelming numbers of monovalent ions. After rinsing, the exchanger is again ready for use. The regeneration process can take from a few minutes to as much as an hour. The process can operate fully automatically (regeneration initiated by a timer or water meter), semi-automatically (manual start for regeneration), or manually. The method of regeneration must be considered based on the frequency of "operator checks" and the amount of usage.

(c) Efficiency. Modern ion exchange media are quite durable and can be regenerated an almost unlimited number of times. The frequency with which regeneration is needed is a function of the capacity of the medium (i.e., the theoretical mass of ions that can be exchanged per unit volume of medium), the hardness of the water, the flow rate, and the efficiency of the regeneration process. Since regeneration is essentially a mass action phenomenon, there is a decreasing rate of return for increasing the regenerate concentration. Thus, it may be economical to regenerate with a weaker solution and do so more often, than to use a more concentrated solution and achieve a greater percent recovery of the surface active sites. However, small system design may be controlled more by operational convenience than by economics. The desired efficiency and frequency of regeneration are key design factors.

(2) Exchange capacities. The history of ion exchange is such that the commonly used units of expression may be

confusing. For example, the capacity of a given type of exchange medium is often expressed as so many grains of hardness per cubic foot, and manufacturers' literature may express concentrations in grains per gallon. One grain is equal to 1/7000 pound or about 0.0684 gram. One grain per gallon is approximately 17.1 mg/L.

(3) Split treatment. The nature of the ion exchange process is such that it is not possible to design a unit to remove less than 100 percent of the hardness applied. In practice some bleedthrough may occur, but this is insignificant in the case of potable water treatment. Thus, softeners may be assumed to be completely effective until the medium is exhausted, significant breakthrough occurs, and regeneration is required. Since there is no need to completely soften a public water supply, only a portion of the total flow need pass through the softener. The remainder may be bypassed and then mixed with the softened water to achieve the desired level of hardness. Typical combined product water should have a total hardness of 50 to 100 mg/L as calcium carbonate.

(4) Wastes. The principal difficulty with ion exchange softening of small water supplies is that waste regenerate solution is produced when the medium is regenerated. The volume of this wastewater is relatively small, but a suitable means of disposal must be available. Unfortunately, municipal wastewater systems may not accept this material. The high sodium concentration may make septic tank disposal unacceptable also since, for certain soils, sodium may be exchanged for the naturally occurring multivalent metals usually present and reduce permeability. The significance of this potential problem is quite variable. Disposal of high-strength sodium-recharge water should be coordinated with environmental regulators. If sodium-containing wastewater disposal becomes a problem, then a hydrogen form cationic resin should be evaluated.

6-6. Taste and Odor Removal

Undesirable tastes and odors in drinking water can stem from a variety of sources (Zoeteman, Piet, and Postma 1980). For groundwaters, a common cause of complaint is hydrogen sulfide, which produces a "rotten egg" odor. For surface waters, the problem is usually related to the metabolic activity of algae, actinomycetes, or other organisms, or contact with decaying vegetative matter. Reactions between chlorine and certain organic substances (e.g., phenols) may produce a noticeable taste or odor. Chlorine itself may be objectionable to some users, but in normal concentrations most people become acclimated quickly. Occasionally, taste and odor problems are related to contamination by industrial, municipal, or domestic wastes. This is potentially the most serious situation since some contaminants deleterious to public health may be presumed to be present. Excessive concentrations of iron can produce metallic tastes that are unacceptable to many people. Taste and odor problems are mentioned briefly in paragraph 3-8e.

a. Reporting units. Dealing with tastes and odors can be troublesome and frustrating for water system operators because the problems frequently are of an intermittent nature, the sensitivity of exposed individuals varies greatly, and control or treatment must be geared to the specific cause if success is to be expected. The problem is complicated to a significant extent because there is no reliable test procedure except to empanel a group of individuals to smell and/or taste the water. (Taste and odor are so closely linked that it is usually impossible to separate them. However, some substances, e.g., inorganic salts, may produce a taste without any noticeable odor.) The reporting unit commonly used for odors (and associated tastes) is the TON. TON is defined as the dilution factor required before an odor is minimally perceptible. Thus a TON of 1 (i.e., no dilution) indicates essentially odor-free water (paragraph 6-12*a*).

b. Removal. The design of taste and odor removal processes should not be undertaken until the cause of the problem has been identified and bench or pilot testing has been performed to determine the effectiveness of alternative techniques. Commonly used approaches are discussed below.

(1) Management practices. Often the best control procedures are to develop an alternative supply, or manage the source to minimize the problem. The latter approach is especially appropriate for surface waters when biological activity (algae, decaying vegetative matter, etc.) is the source of the tastes and odors. Typical management practices include aquatic weed control programs such as deepening or varying the water level of reservoirs, dredging to remove growing plants and debris, and chemical treatment. Chemical treatment programs are also very effective against algae. Land use management in the watershed area is an effective tool that may be used to control weed and algae problems by limiting the nutrients entering the body of water to a quantity insufficient to stimulate excessive growth. Control of land use can also be used to protect against contamination by industrial, municipal, domestic, or agricultural wastes.

(2) Aeration. Tastes and odors associated with some substances can be reduced or eliminated by aeration. Surface waters are especially amenable to this method since aeration can usually be integrated into the treatment scheme without great difficulty. The principal removal mechanism is usually stripping, although some oxidation may also occur. Good success with algal metabolites, volatile organic compounds, and hydrogen sulfide is frequently achieved. Even when not totally successful, aeration may substantially reduce the load on other treatment processes such as adsorption. Aerators similar to those used for oxidation of iron (paragraph 6-3*c*(1)) can be used.

(3) Adsorption. Granular activated carbon filters are useful in removing tastes and odors caused by substances that will readily adsorb onto the carbon. Typical units look somewhat like pressure filters, but the term “filter,” which is used by many equipment manufacturers and suppliers, is somewhat of a misnomer since the usual object is to remove dissolved, rather than suspended, substances. However, when filtration is needed, it may be possible to combine the processes. Carbon adsorption is effective against a variety of substances, including organic decay products, residual chlorine and chlorination by-products, pesticides, and some dissolved gases. Synthetic adsorbents are available that are capable of similar or superior performance in specific cases, and that can effectively remove hydrogen sulfide. The sorptive qualities of various carbons and synthetic adsorbents differ substantially. Thus, bench or pilot scale testing is needed before a final design can be accomplished. When the adsorbent is finally exhausted, replacement is necessary since onsite regeneration is impractical for most small systems. In very low flow applications, disposable cartridge type units may be sufficient. Flow rates through granular activated carbon beds generally range from 8 to 20 L/min/m^2 (2 to 5 gal/min/ft^2) of nominal bed surface area. Backwashing is needed occasionally to dislodge solids that may accumulate. Since activated carbon is very effective for dechlorination, it will generally be necessary to disinfect following adsorption. For surface water treatment, application of powdered activated carbon may be feasible. In this process the carbon is added to the water, mixed with it, and then allowed to settle. Filtration is required to remove fines. This approach is especially good when the taste and odor problem occurs only at certain times of the year. Little added expense is involved, except that directly associated with carbon addition, since settling and filtration are usually used for surface water treatment anyway.

(4) Oxidation. Oxidizing agents such as chlorine and potassium permanganate also find application in taste and odor control. However, before the decision is made to use chlorine, consideration should be given to the need for subsequent dechlorination and the possible production of undesirable chlorinated organic compounds. In addition, adequate detention time is needed to ensure process effectiveness. Feeding chlorine for taste and odor control may be accomplished by solution feeders such as the hypochlorinators used for disinfection (paragraph 6-2*b*(5)). When chlorine is used to control sulfides, insoluble precipitates may be formed that can be effectively removed by pressure filters similar to those used for iron removal (paragraph 6-3*c*).

6-7. Stabilization and Corrosion Control

Stabilization and corrosion control are closely related topics and will, therefore, be discussed as a unit.

a. Stabilization. A water is considered stable if it tends neither to deposit nor dissolve solid calcium carbonate. For a given water, stability is a function of the calcium ion concentration, total alkalinity, and pH. Lime may be added to adjust (increase) all three variables simultaneously and is frequently used for this purpose. Other chemicals including sodium carbonate (soda ash), sodium hydroxide (caustic soda), and carbon dioxide are occasionally used. As a general rule, the desirable characteristics of a finished water include calcium and alkalinity concentrations that are similar and in excess of 40 mg/L as $CaCo_3$, a pH no higher than about 9.0 to 9.3 (depending upon the magnesium concentration), and the potential to deposit 4 to 10 mg/L of $CaCO_3$. A thin, hard layer of this precipitate makes an excellent coating that protects the insides of pipes, pumps, hydrants, etc., from corrosion. A water that dissolves $CaCO_3$ is considered corrosive. A water that deposits excessive amounts of $CaCO_3$ may clog pumps and appurtenances and reduce the carrying capacity of pipelines substantially. Thus, it is important to produce a reasonably stable water. The magnesium concentration is important because magnesium hydroxide ($Mg(OH)_2$) tends to deposit in excess in hot-water lines and appliances if the concentration is greater than about 40 mg/L as $CaCO_3$. These deposits dramatically affect the performance and expected life of such items as water heaters. Detailed discussions of stabilization and simplified methods for estimating required chemical doses are widely available (Benefield, Judkins, and Weand 1982; Merrill and Sanks 1977a, 1977b, 1978; and Sanks 1978).

b. Corrosion control. In general, it is not possible to completely protect a water system from corrosion. However, careful control of water quality can reduce the rate at which corrosion occurs quite substantially.

(1) Water quality. Corrosion is usually associated with the following factors. It should be noted that these factors are often interrelated.

(a) Low pH.

(b) Low mineral content.

(c) Low alkalinity.

(d) High dissolved oxygen concentration.

(e) High carbon dioxide concentration.

Most groundwaters do not contain high concentrations of oxygen. Therefore, it is best to limit the opportunity for the water to pick up oxygen as it is pumped, processed, and distributed.

(2) Neutralization. The usual cause of corrosiveness of groundwater is the presence of high concentrations of carbon dioxide and the resulting low pH. While it is possible to strip most of the carbon dioxide out by aeration, this is usually not practiced for groundwaters because the oxygen introduced in the process also promotes corrosion, and double pumping may be required. A better approach is to neutralize the excess carbon dioxide chemically. Either sodium bicarbonate or sodium carbonate (soda ash) is useful for this purpose since they are readily available, are relatively inexpensive, are highly soluble, do not add hardness, produce no residue, and are fairly safe to handle. The neutralizer solution may be injected into the flow, or directly into the well using the same type of feed equipment used for hypochlorites (paragraph 6-2*b*(5)). In fact, the solution may be mixed with hypochlorite and fed simultaneously if desired. This practice reduces equipment costs, but the mixture must be prepared carefully to ensure proper dosage of both chemicals. Application directly to the well offers some potential advantages where it is practical, since protection will be afforded to pumps and other submerged equipment. Other chemicals such as sodium hydroxide (caustic soda), calcium hydroxide (hydrated lime), or calcium oxide (quick lime) may also be used, but may pose operational problems or require special equipment (e.g., lime slaker). Use of sodium bicarbonate or soda ash will increase the sodium concentration of the water. Whether this is undesirable will depend upon the background sodium concentration and the intended use for the water. Excessive sodium intake does appear to be related to hypertension and associated cardiovascular problems, at least for some individuals. An alternative method of neutralization is to pass the water through a bed of clean limestone chips. The corrosive water will slowly dissolve the chips and thus be neutralized. This process can be troublesome since the chips will need occasional replacement, backwashing will be occasionally required to keep the bed from clogging, and a relatively long contact time is required to achieve neutralization. In any case, the design of neutralization processes should be preceded by laboratory experimentation and evaluation to establish required doses, contact times, optimal solution strengths, etc.

6-8. Turbidity Removal

Turbidity (paragraph 3-8*c*) removal is almost always required when a surface water source is used, but is almost never needed for groundwaters. The particles that cause turbidity (mostly clay) are very small (1-200 mm effective diameter) and are affected much more by surface chemical and electrical phenomena than by gravity. Thus, neither settling nor filtration is an effective removal technique unless preceded by treatment to encourage the particles to agglomerate to substantially larger sizes. This preliminary treatment is commonly referred to as coagulation and/or flocculation. The thrust of this section is to alert the reader to important design considerations and not to present detailed procedures. Turbidity removal is covered in almost every text on the subject of water supply engineering. Examples are presented in AWWA (1971, 1990), Benefield, Judkins and Weand (1982), Clark, Viessman, and Hammer (1977), Fair, Geyer, and Okun (1966b), Hammer (1975), Hudson (1981), Sanks (1978), Steel and McGhee (1979), and Weber (1972).

a. Coagulation/flocculation. The exact definitions attached to the terms coagulation and flocculation vary depending upon who is using them. Water chemists generally use the term coagulation to describe the processes that make particle agglomeration possible, and restrict the meaning of the word flocculation to the actual physical agglomeration itself. However, it is often difficult to distinguish the two in an operational setting such as a treatment plant. Therefore, design engineers and water treatment operating personnel often use the terms rather loosely.

(1) Coagulation. Several chemicals, usually called coagulants or flocculants, are available to stimulate the aggregation of smaller particles to a size that can be economically removed from water by settling and filtration. By far the most popular for potable water treatment is filter alum, a commercially available form of aluminum sulfate. Typical water treatment practice calls for sufficient alum to be added to the water to cause the precipitation of aluminum hydroxide. This precipitate, in conjunction with various other aluminum hydrolysis products that also may be formed, acts to neutralize surface electrical charges on the particles that cause turbidity, and enmesh or entrap the particles in the resulting sludge. The process proceeds best if the alum is introduced with intense, short-term (1-minute) mixing, followed by longer term, gentle mixing. What water chemists refer to as coagulation occurs during the brief, but intense, mixing period which design engineers usually call flash or rapid mixing. Energy to provide the mixing usually comes from high-speed mechanical stirrers; however, in-line static mixers are also effective and are useful where it is practical to inject the alum directly into a pipeline. In some cases effective mixing can be accomplished by injecting alum just upstream of the suction side of a centrifugal pump. Alum is commercially available in dry powder, granule, or lump form, and as a liquid. For small plants, liquid alum is usually most practical. Equipment of the same type used for hypochlorites (paragraph 6-2*b*(5)) may be used to feed liquid alum. If purchase of liquid alum is undesirable, the operator can manually mix the dry form with clean water. This requires care since it is important to have a solution of consistent concentration. Alum feed solutions should be mixed at concentrations recommended by the chemical manufacturer. The concentration of the original stock solution affects the aluminum species present.

(2) Flocculation. The longer term, gentle mixing period is called flocculation. The time required for flocculation (i.e., particle agglomeration and enmeshment) to occur is a function

of mixing intensity and the nature of the particles. The mixing must be intense enough to encourage particle collisions, but not intense enough to shear aggregated particles apart. The principal design parameters are the mean velocity gradient, usually given the symbol *G*, and the mixing time, *t*. Optimal performance is usually obtained with *G* in the range of 30-60 L/sec, mixing time varying from 10 to 45 minutes, and $G*t$ (dimensionless) ranging from 10^4 to 10^5. Detailed information for calculating *G* for various mixers are presented in typical water supply engineering texts (AWWA 1971; Benefield, Judkins, and Weand 1982; Clark, Viessman, and Hammer 1977; Fair, Geyer, and Okun 1966b; Hammer 1975; Sanks 1978; Steel and McGhee 1979; and Weber 1972). Mechanical, paddle-wheel type mixers are very commonly used. The coagulant dose necessary to induce flocculation cannot be calculated directly and, thus, must be determined experimentally. The "jar test" procedure, a simple test used for this purpose, is described in texts such as those referenced above and in Hudson (1981).

(3) Coagulation/flocculation aids. Many times experimentation will reveal that the use of a small amount (usually less than 1 mg/L) of some high molecular weight polymeric substance (referred to in the water treatment industry simply as a "polymer" or "polyelectrolyte") in conjunction with alum will substantially improve performance, reduce the amount of alum required, and reduce the volume of sludge produced. Cationic (positively charged) polymers have generally been most effective, but a specific polymer should be chosen only after extensive experimentation. In some cases polymers are used alone (especially for low-alkalinity waters and in conjunction with direct filtration); however, this can be quite expensive. Coagulant aids other than polymers (e.g., bentonite clay and activated silica) are occasionally used. Many water chemical manufacturers provide onsite services to water system authorities at no cost in hope of developing new customers. The manufacturer's representative comes onsite with various polymers and will bench-test various chemicals, polymer aids, etc., at differing dosages to determine optimum performance levels. It is recommended that several water chemical suppliers be invited (at different times) onsite to evaluate their best product and optimal dosage. All information developed should remain privileged to the owner and supplier. This competition will encourage the recommendation of the best chemicals and optimum dosage levels at the most favorable pricing.

(4) Other coagulants. Coagulants other than alum are occasionally used in potable water treatment. Examples include ferric chloride, ferric sulfate, and magnesium carbonate. The ferric salts work in a manner very similar to alum. The use of magnesium as a coagulant requires high pH (10.8 to 11) and is only rarely practiced. Because of the lower costs for these types of chemicals, a polymer manufacturer will usually not test these types of chemicals. If the possibility for usage of lower cost chemicals exists, then an independent evaluation by water treatment engineers or laboratory technicians may be warranted.

(5) pH control. Control of pH is very important when alum is used as a coagulant. The addition of alum to water, and the subsequent chemical reactions that occur, tend to destroy alkalinity and lower pH. This is somewhat unfortunate since the alum coagulation/flocculation process as described in (3) above works best in the pH range from about 5.5 to 8.5. If insufficient natural alkalinity is available to buffer pH into this range, lime, soda ash, or some other substance must be added as a supplement. Fortunately, the optimal pH range is compatible with that required for disinfection by chlorination. Solution feeders such as those used for hypochlorination may be used to add additional alkalinity. Coagulation with the iron salts, mentioned in (4) above, is somewhat less sensitive to pH than is coagulation with alum.

b. Sedimentation. Conventional settling facilities provided at larger turbidity removal plants are often long, narrow (4 or 5 to 1 length to width ratio) rectangular basins with theoretical detention times in the range of 3 to 6 hours at the design flow rate. Most regulatory agencies specify a minimum detention period and a maximum areal (or surface) overflow rate. Typical values for the latter are 20 000 to 30 000 L (500 to 800 gallons) applied per day per square meter of nominal tank surface (or floor) area. For small plants, high-rate settling devices using inclined tubes or plates are very popular. These devices are available in considerable diversity from many equipment manufacturers and suppliers. Both steeply inclined (1 rad (60 degrees) to horizontal) and relatively flat (0.2 rad (10 degrees) to horizontal) designs have been used successfully. The latter are perhaps more common. The use of high rate devices results in reduced space requirements (hence their almost universal use for package type plants) without significant increase in required operator skill, effort, or time. The most important factors in design of settling facilities are to get the water into the basin with a minimum of turbulence, provide an adequate settling period under quiescent flow conditions (never more than a 0.3-m (1-ft) per minute bulk fluid velocity), provide storage for sludge, provide a means to remove sludge periodically, and get the water out of the basin with a minimum of turbulence and reentrainment of sludge. Fulfilling this last requirement generally requires a weir to take the overflow off the tank. Most regulatory agencies insist on a weir overflow rate of not more than 250,000 L (20 000 gallons) per day per meter of weir length. A second weir, or pipe, is usually provided to limit the depth over the outflow weir to that corresponding to the design flow rate. Some solids will, of course, not settle. The velocity of flow in the pipe or channel leading to the filters should be kept low enough (say 0.6 m (2 ft) per second) to keep from shearing these particles into even smaller pieces.

c. Flocculator/clarifiers. A number of equipment manufacturers market combination flocculator/clarifier devices often called solids contact units or upflow tanks. These units incorporate the coagulant feed, rapid mixing, flocculation, and clarification (settling) steps into a single tank by means of pipes, valves, baffles, etc. The principal advantages of this approach are reductions in space, detention time, and piping requirements. An important disadvantage is that the regulatory agency with jurisdiction may find such devices unacceptable. Typical devices are described in detail in AWWA (1971) and Steel and McGhee (1979).

d. Filtration. Sedimentation is less than 100 percent effective for removal of suspended particles, even when preceded by coagulation and flocculation. Thus, filtration is virtually an absolute requirement for any surface water. Detailed graphics depicting various type of filters are presented in most standard texts (AWWA 1971; Clark, Viessman, and Hammer 1977; Fair, Geyer, and Okun 1966b; and Steel and McGhee 1979).

(1) General. The filters most commonly used for small surface water treatment systems are the pressure type. The filter medium most often used is sand having an effective size of 0.4 to 0.6 mm and uniformity coefficient of about 1.3 to 1.7. A 600- to 750-mm- (24- to 30-in.-) deep bed supported by 450 to 600 mm (18 to 24 in.) of hard, rounded, graded gravel (2.4- to 57.2-mm (3/32-in. to 2-1/4-in.) diameter) is common for conventional rapid sand filters. Sometimes as much as one-half the filter bed depth (about 380 mm (15 in.) is occupied by crushed anthracite coal or activated carbon having a uniformity coefficient of 1.8 or less and an effective size of 0.6 to 0.8 mm. Units such as these are called dual media filters. Occasionally three or more different types of media may be used in what are known as mixed media filters. The rationale behind dual and mixed media filters is that the backwashing process distributes the sand in a rapid sand filter such that the smallest grains move to the top of the bed and the largest grains move to the bottom. Since a downflow operational mode is used during filtration, the water encounters the smallest grains (and openings) first. Therefore, most particles are removed in a narrow band near the top of the bed. This causes head loss to build relatively rapidly. If larger grains of some less dense material are also placed in the bed, backwashing will move them to the top of the bed. Thus, while each "layer" is still stratified with its smallest grains at the top, the overall effect is larger grains underlain by smaller grains. With this arrangement, the filter can operate longer at a given flow rate before backwashing is required, or the filter can work at a higher rate without significant loss in product water quality, or perhaps both, compared to a conventional rapid sand filter. The mixed media filter (three or four different densities) is simply a logical extension of the dual media concept. Since the filters represent the last barrier to suspended contamination before the water enters the distribution system, regulatory agencies tend to hold to very conservative design criteria. Typical flow rates are 80 L/min/m^2 (2 gal/ min/ft^2) of nominal filter area for rapid sand filters, although double or triple this value is frequently allowed for dual or mixed media filters. There are several optional methods of filter flow control. For small systems where only one filter is active at a time, some type of constant rate of flow controller on the effluent line works well. The venturi controller is a proven design. At least two filters should be provided. A backwash system capable of delivering treated water at a flow rate of 600 to 800 L/min/m^2 (15 to 20 gal/min/ft^2) of nominal filter area for about 20 minutes is needed to clean the filters. Some filter designs (mostly proprietary) employ other backwashing techniques that may have different requirements. Regulatory agencies generally have very specific requirements with respect to piping, under-drains, backwashing, etc., as well as filtration rates and filter media. Major design factors include filtration rate, filter media, regulatory requirements, desired effluent quality, allowable head losses, and backwashing frequency. As a general rule, effective backwashing is the key to successful rapid sand, dual media, or mixed media filtration. Logsdon and Fox (1982) and Trussell et al. (1980) pertain directly to filtration and may be of interest in addition to the general turbidity removal references cited above.

(2) Slow sand filters. Slow sand filters are generally not used following coagulation, flocculation, and settling. However, for very high quality surface waters, and in cases where groundwaters must be filtered, slow sand filters may be a good choice. Their use for community water supplies may, however, require special regulatory agency approval. Slow sand filters operate by gravity, as do rapid sand filters, but they are never backwashed. When the head loss through a slow sand filter becomes excessive (the exact value depends on the particular design), the filter is taken off line, drained, and allowed to dry. The schmutzdecke (layer of debris, trapped particles, etc.) is then removed, or the surface is at least raked, and the filter is returned to service after subsequent development of a new schmutzdecke. Several cleanings can be performed before the replacement of media is required. The active part of a slow sand filter is the schmutzdecke and the top 25-50 mm (1-2 in.) of sand. The remaining sand acts mostly as a backup or factor of safety, although a few particles may be removed deep within the bed. Thus, before the filter can be returned to service, it must be "ripened." This is accomplished by loading the filter and either wasting the effluent or recycling it to another filter until a new schmutzdecke is developed. Thus, at least two filters, and preferably three, should be provided. A typical filter is composed of a 1220-mm- (48-in.-) deep bed of homogeneously packed sand, having an effective size of 0.2 to 0.4 mm and a uniformity coefficient of 2.5 or less, supported by 250 to 300 mm (10 to 12 in.) of graded (5 to 76 mm

(3/16 in. to 3 in.) in diameter) gravel. Each filter should be equipped with a head loss gauge, an adequate underdrain system, and a cover to prevent algae growth in the water being filtered. Typical flow rates are around 3.26 to 3.67 L/min/m^2 (0.08 to 0.09 gal/min/ft^2) of nominal filter surface area. A water depth of about 1.2 m (4 ft) above the sand surface is typical. A rate-of-flow controller may be used on the filter discharge to ensure a more or less constant production rate throughout a filter run (may last several days to a month or more). Slow sand filters work best when the average raw water turbidity is 10 NTU or less; however, occasional peak turbidity up to 25 NTU can be handled. The filters should not be used for waters containing more than about 0.3 mg/L iron or 0.05 mg/L manganese.

(3) Direct filtration. The direct filtration process (AWWA 1980b; Logsdon and Fox 1982; McKormick and King 1982; and Trussell et al. 1980) is mostly of interest for low-turbidity waters. In direct filtration, coagulants are used, but the sedimentation step (and sometimes even a portion of the flocculation step) is omitted. Thus, the filter is the sole means of suspended solids removal. Direct filtration is a relatively new process and may not be acceptable to many regulatory agencies, especially for small plants. The process is most applicable when raw water turbidity is consistently 10 NTU or less. In these cases the only coagulant required may be one of the polymers previously discussed (*a*(3) above). Direct filtration is accomplished using equipment similar to a rapid sand filter.

e. Package plants. The use of a "package plant" should be fully investigated in situations where surface water treatment is required. Package plants are preengineered, usually prefabricated, treatment plants available in a variety of sizes (40 L/min (10 gal/min) to several million liters per day) from several manufacturers. In many cases they can be delivered to the site virtually intact, set up, connected to an electrical power source and the required piping, and placed into operation in a matter of days if necessary. As a general rule, the technology used is proven, and excellent performance can be expected provided that the manufacturer's recommendations with respect to operation and maintenance are rigorously followed. Some package plants are equipped with process control systems that automatically adjust chemical doses based on raw and finished water turbidity and pH, monitor flow rates, indicate equipment breakdowns, etc. Such a system, when properly maintained, can reduce operator time and result in more efficient use of chemicals. Since the units are preengineered and pre- fabricated (to varying degrees), they can be considerably less expensive than equivalent custom designed and constructed facilities. Most manufacturers have technical support personnel that can work with clients to adapt their units for special local conditions. The industry is highly competitive, however, and a particular plant should not be selected without careful consideration of the design, maintenance requirements, possibly exaggerated advertising claims, and technical trouble shooting support offered by the manufacturer or supplier. Most package plants designed for surface water treatment applications employ mechanically mixed flocculators, high-rate settlers (tube or plate type), and gravity filtration devices similar to rapid sand filters. Solids contact units and pressure filters are offered by some manufacturers. Considerable information concerning package plant performance and cost is available in the literature (Clark 1980; Clark and Morand 1981; Hansen, Gumerman, and Culp 1979; Morand et al. 1980; and Stevie and Clark 1980). In most cases where package plants have failed to produce an acceptable finished water, the fault has been inadequate operation and maintenance, not plant design. For package plant projects, the manufacturer should be required to provide an appropriate amount of onsite training and system operation instructions, separate from operation and maintenance manuals, to the system owners' potential operator.

f. Waste disposal. Typically, turbidity removal results in two waste streams, filter backwash water and sludge. The two wastes are actually very similar except that the former is much more dilute than the latter. In some cases, especially for some types of package plants and in the case of direct filtration, only one "composite" waste stream is produced. Principal components of both types of wastes are the suspended particles removed from the water along with coagulant precipitates that are formed. As a rule, the wastes are not particularly objectionable in terms of odor. If a municipal sewer is available, it may be possible to dump filter backwash water and sludge directly to the waste treatment system. Where both filter backwash and sludge are produced, it may be possible to reduce the volume of the waste substantially by recycling the backwash water to the plant and ultimately disposing of all captured solids in the sludge. It may also be possible to hold filter backwash and sludge in a thickener and haul the thickened sludge away occasionally. It is usually not acceptable to return water treatment wastes to the water source (Reh 1980).

6-9. Total Dissolved Solids Removal

On rare occasions, available sources of water will contain excessive amounts of total dissolved solids (TDS). This problem is most likely to be encountered in groundwaters found in the midwestern and southwestern United States and in surface waters (and some groundwaters) in coastal areas. The chemical species that contribute most frequently to TDS are calcium, magnesium, sodium, bicarbonate, chloride, and sulfate. Unfortunately, it is very difficult to establish fully rational maximum acceptable concentrations for TDS because the various chemical species that may be involved have different effects. In addition, public acceptance of high-TDS waters is quite variable. Common complaints include a salty taste and laxative effect. When possible neither chloride nor sulfate should exceed 250 mg/L and TDS should be no more than 500 mg/L. These values correspond to the maximum contaminant levels recommended by the USEPA pursuant to the SDWA (paragraph 3-5). Methods for removal of TDS include ion exchange and several membrane processes. Among the latter category, reverse osmosis (RO) appears to offer the best prospect for small water systems.

a. Reverse osmosis. When high-TDS water is separated from fresher water by a semipermeable membrane, the natural tendency is for the fresh water to diffuse through the membrane as if it were under pressure, and dilute the high TDS. This hypothetical pressure is called the osmotic pressure, and the overall process is known as osmosis. If sufficient pressure is applied on the high-TDS side of the membrane, the process can be reversed and water from the high-TDS region will diffuse through the membrane and thereby be purified. Thus, fresh water or permeate is produced. This process, reverse osmosis, has been developed for small water systems by equipment manufacturers.

(1) Typical units. Commercially available reverse osmosis units vary mostly in the pressures and membrane materials used, and are suitable for flow rates of a few hundred to a million liters per day. Many designs are modular in concept and can be put together readily to treat much larger flows. A typical unit is composed of a high-pressure pump (1400 to 10 000 kilopascals (200 to 1500 pounds per square inch)) and a membrane module. Several membrane materials including nylon and cellulose acetate are available. In the typical unit the membrane is in the form of bundles of hollow fibers. Major factors to consider in design are first cost, operation and maintenance costs (which include pumping and membrane replacement), feed water quality, temperature, salt rejection (i.e., effectiveness of the membrane in containing the dissolved solids), water recovery (i.e., efficiency with respect to permeate production to feedwater flow), waste disposal, and required pretreatment. The last category is very important since hardness, iron, manganese, organic matter, sulfides, and chlorine may tend to foul or damage membranes. Proper pretreatment (which obviously can be extensive and expensive) is probably the single most important factor in successful RO treatment. This can be especially important for units with low water recovery (the typical range is from 20 to 95 percent). If water recovery is 50 percent, for example, the pretreatment units must be sized for a flow rate twice the actual production rate.

(2) Efficiency and waste disposal. Salt rejection rates vary considerably, but 90 to 99 percent removal is not uncommon. An exception is nitrate, which may typically be removed with an effectiveness of only 50 to 80 percent. The reject water from an RO unit may thus contain around 90 percent of the total feed water TDS in a flow that can vary from 5 to 80 percent of the feed water flow. Disposal of these wastes can be a serious problem and should be considered early in the design process. In most cases the disposal method must be approved by both water and wastewater oriented regulatory agencies. When designing RO facilities, it is important to work closely with equipment manufacturers and suppliers since they are a major source of basic information, and common practice is to purchase preengineered, manufactured units ready to install. Most are available complete with automatic control systems. Many water supply texts present discussions of RO, as well as other membrane processes (ultrafiltration, dialysis, and electrodialysis) that may be of interest (Clark, Viessman, and Hammer 1977; Fair, Geyer, and Okun 1966b; Hammer 1975; Lehr et al. 1980; Sanks 1978; Steel and McGhee 1979; and Weber 1972). The quality of the feed water has a major impact on production rate; thus, expected variations in raw water quality must be considered. Temperature is also important, with higher solvent recovery, but shorter membrane life, associated with higher temperatures. pH can also be important and, depending largely upon the specific membrane chosen, it may be necessary to adjust the feed water pH.

b. Ion exchange. Ion exchange may also be used for TDS removal. The process is similar to that previously described for softening (paragraph 6-5*b*) except that both cationic and anionic exchange media are used. Since removal of all TDS is usually desired, hydrogen form cationic media and hydroxide form anionic media are normally used. The former may be regenerated with strong acid and the latter with strong base. Major operational problems are associated with pretreatment, regeneration (the solutions are very corrosive), waste disposal, and limited durability of most types of anionic media. The advice of equipment manufacturers and suppliers should be heeded when selecting ion exchange devices and media.

6-10. Color Removal

True color (i.e., color due to dissolved substances) is often very difficult to remove. Apparent color (color due to suspended substances) is generally removed along with turbidity (paragraph 6-8). A brief discussion of color is presented in paragraph 3-8*d*. True color can occasionally be removed by oxidation or adsorption in a manner somewhat similar to removal of iron and manganese (paragraphs 6-3 and 6-4) and tastes and odors (paragraph 6-6). Color problems can sometimes be controlled at the source if the precise cause can be determined. Paragraph 6-11, which deals with removal of dissolved organic substances, may be relevant to color removal as well. Since the presence of true color may indicate industrial contamination, the source of any color problem should be fully identified.

6-11. Control of Organic Substances

Organic substances may contribute to a variety of problems including taste, odor, and color, and some are known to have adverse health effects. It is known that hundreds and perhaps thousands of organic compounds may be present in any given natural water, even groundwater previously thought to be relatively uncontaminated. The real environmental and public health significance of most of these substances remains unknown, however. This is especially true of long-term effects of exposure to the very low concentrations typically found in water supplies. Considerable attention has been focused on pesticides and on one group of chlorinated hydrocarbons called trihalomethanes (e.g., chloroform), and more recently halo-acetic acids (HAA). In many, perhaps most, cases the removal of organic compounds, especially dissolved compounds, from drinking water is expensive, requires skillful operation, and can be monitored only with the aid of complicated and expensive analytical techniques. This type of activity is not readily compatible with typical small water system operation. Recognizing the difficulty and expense in monitoring very low concentration, the USEPA provides for specific techniques for water treatment in the case of detected contaminants.

a. Trihalomethanes. Precursors of trihalomethanes (THMs) exist in most natural waters and are converted to THMs by halogenation, for example by chlorination as usually practiced at water treatment plants. A maximum contaminant level for THMs is specified in the drinking water standards. Control of THMs and their precursors is a newly developing field. However, several possible approaches have already proven somewhat effective. These are included as BAT processes and would include the controls described below.

(1) Watershed management. The best approach to control of THMs, and all organics for that matter, is to find a water source that does not contain significant concentrations of them, and then protect that source from subsequent contamination by careful watershed management. This is not a feasible approach for most larger cities, but can be very practical for small communities. Control of land use in the watershed area can be very effective against synthetic chemicals of industrial or agricultural origin. In addition, control of the algal population in the reservoir (if there is one) can be of major importance since algae are responsible for some THM precursors.

(2) Conventional treatment. In many instances THMs can be effectively controlled by eliminating precursors (prior to chlorination) by strict attention to the conventional treatment processes such as coagulation, flocculation, settling, and filtration. Optimization of the performance of these processes, coupled with no chlorination of untreated water, will often suffice. When the situation dictates that raw waters be disinfected, some method other than chlorination (e.g., ozonation or chlorine dioxide treatment) can be used. The effectiveness of this approach is widely documented (Kavanaugh et al. 1980; Singer et al. 1981; and Vogt and Regli 1981).

(3) Alternative disinfection. Disinfectants other than chlorine may be used in water treatment. While this will solve the problem of formation of chlorinated organics during treatment, there may be undesirable side effects, including increased costs and lower residual disinfecting power. Alternative disinfectants are introduced in paragraph 6-2*a* (see also Hoff and Geldreich 1981 and Rice et al. 1981).

(4) Aeration. Volatile organic compounds can sometimes be transferred from the liquid to the gaseous phase and removed from water by aeration. The process is similar to that described for iron and taste and odor problems, except that packed tower aeration systems are more predominant than aeration trays (paragraphs 6-3*c*(1) and 6-6*b*(2)). Care must be used to avoid picking up contaminants from the air (Kavanaugh and Trussell 1980).

(5) Chemical oxidation. In some cases, THM precursors can be removed by chemical oxidation in concert with more conventional processes. Permanganate and ozone may be useful for this purpose (Glaze et al. 1982; Peyton et al. 1982; Rice et al. 1981; and Singer, Borchardt, and Colthurst 1980).

(6) Adsorption. THM precursors and many other dissolved organic compounds may be removed from water by adsorption. Most applications of this methodology have employed either granular or powdered activated carbon, but synthetic media have also been used (Boening, Beckmann, and Snoeyink 1980; Cannon and Roberts 1982; Suffet 1980; Krabill 1981; and Weber and van Vliet 1981). Granular carbon is usually used in pressure operated contactors similar to ion exchange units, while powdered carbon is mixed with the water and subsequently removed by settling and filtration. Synthetic media are usually employed in the same manner as granular carbon.

b. Other organics. Any of the thousands of organic chemicals used daily in industrial, commercial, municipal, and domestic activities may wind up in a public water supply. The sheer numbers and diversity of the possible organic contaminants make the problem of removing them a difficult one. The current list of synthetic and volatile organic carbons is quite extensive and continues to grow. The techniques mentioned in *a*(1), *a*(2), *a*(4), *a*(5), and *a*(6) above are useful in dealing with many organics other than the trihalomethane precursors (Dyksen and Hess 1982 and Love and Eilers 1982). However, the nature of organic contamination is such that no removal process or method should be included in a water system design until the contaminants and their sources are identified, and the method or process has been tested at the laboratory or pilot scale and proven effective. Maximum contaminant levels are subject to enforcement; therefore, it behooves the designer to contact the appropriate drinking water regulatory agency for discussion and current requirements. Many times the experience of regulators with specific systems and contaminant problems is exceedingly useful.

6-12. Membrane Technologies

Recent improvements in membrane technologies have allowed more versatile applications of drinking water treatment for small systems. Previously, more or less, membranes were used in drinking water treatment for desalting brackish water and seawater. Membranes are finding more applications in filtration and disinfection compliance. Beside reverse osmosis (RO) as discussed in paragraph 6-9, "Total Dissolved Solids Removal," engineers classify membranes in three additional categories: microfiltration (MF), ultrafiltration (UF), and nanofiltration (NF). Depending on the water treatment need, membranes have a particular processing function. Membrane systems can be used for removing particles, microorganisms, natural and synthetic organic matter, and inorganic chemicals. Though the processes and equipment operations have improved over the years, whether or not to employ a membrane unit operation remains dependent largely on the treatment compliance criteria, chemical and physical condition of the source water, and whether the operations and maintenance personnel are adequately staffed and trained. Better understanding of membrane filtration for water treatment is required before universal application can be assumed. Among the considerations for additional research include pretreatment and membrane fouling, precursor removal, and preoxidation issues. However, as the technology and systems continue to improve, membrane technology may offer an attractive alternative for treatment and should be considered in the overall evaluations.

Chapter 7
Pumping, Storage, and Distribution

7-1. Introduction

The various components of a water supply system should be designed to work together effectively and efficiently to ensure that sufficient water is available to meet variable rates of demand. This is especially true of smaller systems since maximum demand rates are often many times greater than average rates. In order to accomplish the dual goals of effectiveness and economy, the design process must be a carefully integrated activity. Hence, the implications of the design and operation of each component on the design and operation of every other component should be considered. The relationships among the pumping, storage, and distribution functions are especially important and are, therefore, considered together in a single chapter. In reality all three must be considered essentially simultaneously; but for the sake of clarity, pumping will be discussed first, storage second, and distribution third.

7-2. Pumping

It is almost never possible to remove raw water from its source, process it, and deliver potable water to the ultimate users by gravity flow alone. Thus, pumping is almost always required. However, for many small water systems (e.g., single well with relatively high yield), only one pump may be required. On the other hand (e.g., surface water requiring substantial treatment), several different pumps may be needed. Regardless of the application, the procedure to be followed in selecting pumps and designing pumping facilities is essentially the same.

a. Selecting pumps. Pump selection is discussed in many water supply textbooks, speciality handbooks, and manuals. Examples include Campbell and Lehr (1973), Clark, Viessman, and Hammer (1977), Daffer and Price (1980), Hicks and Edwards (1971), Linsley and Franzini (1979), Merritt (1976), Salvato (1982), Sanks (1978), Steel and McGhee (1979), USEPA (1974), Walker (1976), and Wright (1977). Guidelines, specifications, and standards for pumps are issued by a number of agencies and organizations including the Department of the Army (EP 310-1-5) and AWWA. References that may be especially helpful to designers of small systems include the Manual of Individual Water Supply Systems (USEPA 1974), Manual for Safety Rest Area Water Supply Systems (Folks 1977), Environmental Engineering and Sanitation (Salvato 1982), and Pump Selection: A Consulting Engineer's Manual (Walker 1976). A brief discussion of pumping requirements applicable to military (Army and Air Force) installations is presented in TM 5-813-1.

(1) Data requirements. It is not possible to select the best pump for a given application until the expected operating conditions are fairly well defined. Thus, design (at least preliminary design) of distribution and intake piping must proceed pump selection. Consideration of storage requirements may proceed more or less simultaneously with pump selection. The following specific information must be available:

(a) Maximum safe rate at which water can be supplied to the pump (e.g., well or reservoir yield).

(b) Average and maximum rates at which water must be delivered by the pump to the distribution/storage system (this requires knowledge of the type and volume of storage that will be available) .

(c) Minimum available net positive suction head (this requires knowledge of the maximum lift required and all head losses on the intake side of the pump).

(d) The range of discharge heads the pump must work against (this requires knowledge of the system head/flow characteristics, which include the effects of all head losses on the discharge side of the pump and the maximum and minimum allowable pressures in the system).

(e) Characteristics of the water to be pumped (e.g., temperature, sand content, corrosiveness).

(f) Availability of suitable electric power at the site.

(g) Expected level of operation and maintenance capability (i.e., operator time per day, skill level of operator, availability of maintenance and repair support).

(h) Desired placement of pump (e.g., indoors, outdoors, submerged, in a dry well).

(i) Design period.

Once these and perhaps other site-specific factors are known, it is possible to consult manufacturers' literature and consider the available pumps. A major portion of this process involves consideration of trade-offs among the reliability, first cost, and operation and maintenance cost of various pumps having suitable flow/head/efficiency characteristics.

(2) Types of pumps. Several kinds of pumps are available, but centrifugal pumps are almost always chosen for deep well or surface-supplied water systems. In the latter case, either horizontal or vertical pumps may be used. The choice depends largely on the type of intake and storage systems used

and the desired placement of the pump. For deep well applications, vertical turbines or submersible pumps are usually used. Both types are actually multiple-stage (stages stacked vertically) centrifugal pumps. They differ in that for the vertical turbine type, only the pumping head is submerged, while for the submersible type, the pumping head and driver are closely coupled and the entire unit is submerged. Vertical turbine pumps offer some extra convenience since the driver is easily accessible for maintenance or replacement, but require a drive shaft to connect the driver to the pumping head. Therefore, the well must be aligned well enough to accommodate the shaft. Submersible pumps can be installed in poorly aligned wells so long as there is sufficient clearance to lower the pump to the desired depth. Detailed discussions of the advantages and disadvantages of various types of pumps and the factors to consider when choosing among them are readily available in the literature (e.g., Folks 1977; Hicks and Edwards 1971; USEPA 1974; and Walker 1976) and are not reproduced herein. The pump chosen should conform to EP 310-1-5 and/or AWWA.

(3) Operating reliability. Regardless of project size, economic considerations are important in pump selection. However, the very nature of small water systems puts a premium on minimizing operational difficulty and expense. As a result, it is usually best to use a pump and control system that is simple, rugged, and reliable even though less expensive (first cost) options may be available. For this reason, constant speed units are usually preferred. Whenever feasible, pumps and drivers should be selected that will operate near their peak efficiencies under the actual operating conditions that are expected. Maximizing the efficiency of pumps and drivers (subject to the constraints of operational ease and reliability) will tend to reduce operating costs without reducing dependability significantly, when compared to overdesigned facilities.

(4) Overdesign. Inefficiencies arising from overdesigning (i.e., choosing a pump that will, for a given head, deliver more water than is needed) are common since both engineers and manufacturers' representatives tend to be "conservative." The result of "conservative" design is often a system that operates inefficiently because it is capable of delivering more water than is ever required. Such systems are wasteful in terms of both initial investment and continuing operating cost. To avoid this pitfall, designers must consider pump characteristics and system head curves carefully and work closely with manufacturers' representatives (Daffer and Price 1980). In this regard the efficiency of both the pump and the driver should be considered. Fortunately, electric motors are usually fairly efficient over a broad load range (e.g., 50 to 125 percent of the rated capacity). However, in the smaller sizes, high-efficiency motors may be as much as 10 percent more efficient than their standard counterparts. At typical electrical power rates, such motors are likely to be a good investment. Pumps with fairly steep characteristic curves are usually preferred since their capacity to deliver water is relatively unaffected by changes in head. As pumps, water lines, valves, etc., age, head losses will tend to increase. This can significantly affect flow rates if both the pump characteristic and system head curves are fairly flat.

b. Pumping stations. Pumping stations should protect pumps and other equipment from weather and vandalism. They should be located on high ground (e.g., 0.3 m (1 ft) above the 100-year flood level), or protected by adequate earthwork. Floors should be raised at least 150 mm (6 in.) above finished grade and adequate interior drainage should be supplied. In the case of well houses, the floor should be sloped to direct drainage away from the well. Freeze protection, including adequate insulation and heaters, should be provided as should ventilation to prevent the overheating of equipment during warm weather. Care should be taken to ensure that neither raw nor treated water can be contaminated by lubricants, maintenance materials, insects, birds, small animals, etc. Where architecturally acceptable, windows should be kept to a minimum and a security-type fence should be provided to discourage unauthorized entry. Pumping stations should be large enough to allow free access to all equipment and to facilitate maintenance work. Repairs that are technically quite simple can be made very complex by poor placement of pipes and equipment and insufficient room to maneuver. It is good practice to go over pipe layouts and equipment and valve placement with experienced operators before deciding on a final design. Another good approach is to assume that sooner or later every piece of equipment, pipe, or fitting will fail, and then consider what will have to be done to make the necessary repair or exchange. Special attention should be given to ensuring that craneways, hoist beams, eye bolts, or double doors are provided to allow for removal and replacement of heavy items such as pumps, motors, or tanks. When the pumping station doubles as a treatment facility, room must be allowed for chemical storage and laboratory activities as well as the treatment units. In some cases, e.g., gas chlorination, a separate room must be provided. Generally, the local or state regulatory agency will have a number of specific requirements relative to pumping stations.

c. Piping and appurtenances. Each pump should be equipped with a pressure gauge and flowmeter on the discharge line so that performance can be monitored. The piping should be arranged to result in minimal head losses, and valves should be located so that each pump can be completely isolated when necessary. Where multiple pumps are used, each one should have its own intake, or the multiple intake should be carefully designed to ensure that all pumps have essentially the same inlet conditions. Care must be exercised to make sure that the pumps always draw water, not an air/water mixture, or air alone. The specific locations of check valves and other appurtenances will depend on the inlet conditions, type of storage, piping layout, and regulatory requirements. Provisions for sampling and possible future chemical addition locations should be considered in the final design.

d. Capacity. Where feasible, at least two pumps, each having capacity equalling the required demand, should be provided. Common practice is to have the pumps alternate in service. For multiple-pump systems, at least one pump should be capable of meeting the average demand and the remaining pumps should have a combined capacity at least equal to the average demand. Where fire protection is afforded, other requirements may be imposed (paragraph 4-7*b*(1)(c)). For well systems, if practical, it is good to have at least two wells (and pumps) with each one capable of meeting the average demand in a fraction of a day (e.g., 16 hours of operation). However, for small systems, this may be impractical. In such cases, a spare pump and motor should be available. Multiple pump and complex control arrangements relying on various sizes of pumps to meet varying demand for water are usually not practical for small systems. Generally, it is better to use identical pumps in alternating service and meet higher rates of demand from storage or by longer and/or more frequent pumping cycles.

e. Emergency operation. As a general rule, some type of emergency operating capability should be maintained. The relative importance of such a capability is, of course, a function of the local situation (i.e., type of water service provided, storage capacity, the ramifications of interrupted service). For small well systems and small surface water systems, it is usually more practical to provide emergency electrical power by a gasoline or diesel fuel powered portable generator. Deep well systems, depending on importance of the operation, may have permanent fuel powered or dual drive pumps installed. In some cases, local regulations may require the capability for temporary/emergency power connections.

f. Lightning protection. Electric motors should be provided with some type of protection from "near miss" lightning strikes. This is especially true for submersible pumps. It is virtually impossible, however, to provide protection from a direct hit. The best source of information concerning lightning protection is usually the utility company providing the electricity.

g. Pump installation. Pumps should be installed according to instructions provided by the manufacturer. Strict attention should be given to correct anchoring and alignment and to protecting every part of the pump, frame, and driver from loads or stresses (including those of thermal origin) induced by the piping. Failure to observe these precautions can lead to operational problems ranging from excessive vibration and noise to complete failure of the bearings, drive shaft, pump base, or casing.

7-3. Storage

The primary purpose of water storage is to ensure that an adequate supply of water is available at all times. Careful sizing and siting of storage facilities permit the use of economical pipe sizes in the distribution system, reduce the magnitude of pressure variation within the system, can make it possible to operate production facilities at reasonably uniform average rates rather than substantially higher peak demand rates, or can allow production facilities to operate according to some convenient schedule. As emphasized in paragraph 7-1, it is not possible to isolate the design of storage facilities from that of other water supply system components. Thus, design of storage facilities is highly site-specific, and there is no simple procedure that can always be used to size and locate the various tanks that may be required. Rather, the designer must consider the given water supply system as a whole, and choose storage facilities that are compatible with other system components, in order to achieve a total system design that will be both economical and able to serve the intended purpose well. General discussions of water storage requirements are presented in many textbooks and handbooks (Clark, Viessman, and Hammer 1977; Folks 1977; Linsley and Franzini 1979; Merritt 1976; Salvato 1992; Steel and McGhee 1979; USEPA 1974; and TM 5-813-1) and in paragraph 4-3 of this manual. In addition, many state and local regulatory agencies publish guidance for meeting their specific design requirements. In the discussion presented below, primary emphasis is given to treated (finished) water storage.

a. Types of storage. Finished water may be stored in underground, ground level, elevated, or hydropneumatic (pressurized) tanks. In essence, the choice of the type of storage to be used depends upon the purpose for which the water is to be stored, the volume of water that must be stored, topography, climate, the areal distribution of the customers, and economic factors.

(1) Underground and ground-level tanks. Underground and ground-level tanks are usually used for intermediate storage (i.e., following treatment, but prior to entrance into the distribution system), but may also be used for distribution when the topography is such that a beneficial location is available. They are commonly constructed of either concrete or steel. The choice is usually dictated by economic factors.

(2) Elevated tanks. Elevated storage tanks are usually constructed from steel and used primarily for distribution purposes, although at larger treatment plants they may be used to supply water for use in backwashing filters. Elevated storage tanks (and underground or ground-level tanks located at sufficiently high elevations) can supply water for distribution by gravity flow. In addition, they offer certain operational

advantages in that they can be designed to "float on the system." In this type of arrangement both the pump(s) and storage tank(s) are connected directly (but independently) to the distribution system. During periods of high demand, water is supplied from both the pump and the tank. When demand is less than the pumping rate, the tank is gradually filled to some preselected high-water level at which time pump operation ceases. Water is then supplied by the tank alone until the preselected low-water level is reached and pumping begins again. The operation of the pump may be controlled automatically or manually. If the pump and tanks are located properly (usually on opposite sides of the service area), pipeline friction losses can be held to a minimum, even during high demand periods. This saves pumping energy (and possibly capital) costs and reduces the magnitude of the pressure variations in the distribution system. If the volume of the storage tank is large enough in comparison to the daily demand for water, it may be possible to provide an uninterrupted supply, even when it is necessary to make major repairs to pumps or other equipment. When it is not feasible to connect the pump directly to the distribution system, peak demands must be satisfied from the tank alone. This type of operation is less flexible than that described above, but may prove to be completely satisfactory for many small water systems.

(3) Hydropneumatic tanks. Hydropneumatic (pressure) tanks are very commonly used to distribute water in smaller water systems, especially those drawing on a groundwater source. The actual useful storage provided in a hydropneumatic tank is usually quite small in comparison with the nominal volume of the tank. Typical values range from 10 to 40 percent. The former is indicative of situations where all the pressure is supplied by the pump and the latter of designs that include an air compressor to boost pressure. In operation, the pump supplies water to the tank in response to signals from a control system designed to maintain the pressure in the tank between preselected high and low limits. (Note: these control pressures correspond to high-water and low-water levels in the tank.) As the water enters and the level in the tank increases, the air in the tank is compressed and thus the water is stored under pressure. When the pressure rises to the preselected value, the control system automatically shuts off the pump. Additional air may or may not be added, depending upon the particular system design. Water flows out of the tank into the distribution system upon demand. As this occurs, the pressure in the tank drops. When it reaches the preselected minimum value, the pump is automatically activated by the control system and the cycle described above is repeated. Over the course of time, there is a tendency for the water to gradually absorb the air in the tank and, thus, the tank may become "waterlogged." This can be avoided by using a control system that does not allow the water in the tank to rise above the design high-water level in combination with an air compressor to add air as needed. It is also possible for tanks to become "air bound" if too much air is added, or if the water pumped into the tank contains excessive concentrations of dissolved gases. This problem can be avoided if the tank is equipped with a valve that acts automatically to release excess air and a control system that is responsive to both pressures and water levels. Some manufacturers supply hydropneumatic tanks with flexible dividers between the air and water compartments referred to as "diaphragm" or "bladder tanks." This physical separation of the air and water minimizes problems associated with waterlogging and air binding. These tanks eliminate the need for multiple control devices as described earlier to prevent "waterlogging" or "air bound" conditions, thus providing more reliable and maintenance-free service. Thus, this type of tank should be used when available in an appropriate size. Often, up to three tanks in parallel are placed in service to provide adequate storage and system pressure. Hydropneumatic tanks are ideally suited to many small water systems; however, as a practical matter, the pump must be sized to meet peak demand requirements alone, since it is not feasible to provide sufficient storage in the tank to respond adequately to sustained high rates of demand. Occasionally, hydropneumatic tanks are used in concert with intermediate underground or ground-level storage, for example when the source yield is low in comparison to peak demand rates, or to enable economical or convenient operation of treatment facilities. Hydropneumatic tanks are almost never used together with elevated storage.

b. Storage volume. The mass diagram (paragraph 4-3*b*(1)), or some similar approach, may be used to size any storage facility so long as inflow (supply) and outflow (demand) rates are known. In practice, distribution storage volumes are usually determined from consideration of a combination of factors including the ramifications of supply interruptions, the reliability and expected repair frequency of key system components (e.g., pumps), expected time required to make repairs, availability of emergency backup equipment or water supply, regulatory agency requirements, economics, and some type of inflow/outflow analysis. For small systems, economic and regulatory considerations often combine to establish the design storage volume.

(1) Nonpressurized storage. Where it is feasible to use elevated tanks (or underground or ground-level tanks located at sufficiently high elevations) for distribution storage, it is good practice to provide reserve storage for emergencies. Considering the delays that may occur in repairing pumps or other equipment (especially for small rural systems having little in-house repair capability), or in restoring electrical power to rural areas following a major storm, a 2- or 3-day supply is desirable. Ideally, the decision of how much reserve capacity to build into a design should be determined by consideration of the trade-off between losses that would result from an interruption in water service and the cost of reserve

storage capacity. For small water systems, limited investment capital often controls this aspect of design.

(2) Hydropneumatic storage. As a general rule, the nominal volume of a hydropneumatic storage tank should be about 10 times the feeder pump capacity per minute. The following expression may be used to size such a tank more precisely:

$$V = \frac{(Q)(T)}{1 - \frac{P_{\min}}{P_{\max}}} \tag{7-1}$$

where

V = required tank volume, liters

Q = design flow rate, liters per minute

T = desired storage time at flow rate Q, minutes

P_{min} = minimum desired absolute operating pressure (atmospheric), kilopascals (kPa)

P_{max} = maximum desired absolute operating pressure (gauge pressure plus atmospheric pressure), kPa

In common practice, the maximum hourly flow rate is used for Q, T ranges from 15 to 20 minutes, and the pump is designed to meet the maximum instantaneous demand. Many regulatory agencies have very specific rules governing the design of hydropneumatic tanks. An excellent design example is presented by Salvato (1992).

c. Storage tank design. Water storage tanks may be constructed of reinforced concrete, prestressed concrete, steel, or other suitable material, depending upon the function of the tank, economic factors, and regulatory agency requirements. Some specific points to consider are outlined below. Information sources that should be consulted prior to final selection include manufacturer's literature and representatives, applicable AWWA Standards, American Society of Mechanical Engineers Code requirements (primarily for hydropneumatic tanks). U.S. Army Corps of Engineers Guide Specifications, and state and local regulatory agency rules and regulations.

(1) General requirements. All finished water storage tanks should be located and protected such that the contents will not be subjected to contamination resulting from precipitation, surface runoff, flooding, groundwater intrusion, or discharges from storm drains or sewers. Use of single common-wall separation between treated and untreated water should be avoided. Tanks should be covered and all vents and access points should be covered or screened to exclude the entry of birds, animals, insects, airborne dust, etc. Overflow pipes should be provided for nonpressurized tanks and should be terminated near the ground in a way that will prevent the discharge from the overflow from eroding the ground surface. However, overflow pipes should be terminated far enough above the ground surface to prevent the entry of surface water. Some type of access, generally through the top of the tank, should be provided to facilitate cleaning and maintenance. Provisions should be made for securing the covers of all access points to preclude contamination of the contents. Non-pressurized tanks should be vented and the vents should be protected to prevent contamination of the contents. All metal surfaces should be protected by suitable paints or other protective coatings conforming to AWWA Standards or U.S. Army Corps of Engineers Guide Specifications and meeting local regulatory requirements for portable water service. Finished water storage tanks should always be disinfected prior to being placed in service. Allowing a treated water solution containing an initial chlorine concentration of at least 50 milligrams per liter to remain in the tank in contact with all surfaces normally in contact with the water (i.e., up to the high-water level) for at least 24 hours will usually be sufficient. However, the effectiveness of this or other disinfection method should be confirmed by draining the tank completely, refilling with treated water, and carefully analyzing several representative bacteriological samples. The public health agency with jurisdiction will generally have detailed procedures that must be followed for taking and processing the samples. In most cases, these agencies will perform the actual analyses themselves.

(2) Ground-level and elevated tanks. Ground-level and elevated storage tanks should be provided with interior and exterior ladders (with removable bottom sections), water level indicators, sampling taps, and appropriate freeze protection, and should be, to the maximum extent possible, vandal proofed. They should be enclosed by a sturdy fence (for example, 1.8-m- (6-ft-) high chain link with three strands of barbed wire on top) provided with a securable gate. As a general rule, the tank overflow should be located so that the maximum hydrostatic pressure in any part of the distribution system will not exceed 24 m (80 ft) of water. Also, it is good practice to choose a design such that the working elevation of the water surface in the tank will not vary more than 6 or 8 m (20 or 25 ft) during normal operation. For tanks that float on the system, valving should be arranged so that the tank can be isolated and completely drained without causing loss of pressure in the distribution system.

(3) Hydropneumatic tanks. Hydropneumatic tanks are usually cylindrical and may be oriented with the long axis either horizontal or vertical. The former is more common for larger tanks, while the latter is usually used for very small (e.g., individual home or farm) systems. In either case, the tanks should be provided with bypass piping, pressure gauge, sight glass (for viewing the water level), automatic blow-off valve, a mechanical means for adding air, drain, and pump/ pressure/water level control system. It is highly desirable that the entire tank and all appurtenances be located indoors; however, it may (depending upon climate and regulatory agency requirements) be permissible to house only that end of the tank where the pressure gauge, sight glass, controls, etc., are located. The enclosure should be heated and ventilated. This is very important to ensure dependable control system operation. Considerable care should be given to selection of a simple, rugged, dependable pump/pressure/water level control system. It is unreasonable to expect operators of small water systems to be able to make delicate adjustments and repairs to operating control systems.

7-4. Distribution

The purpose of a water distribution system is to deliver water of suitable quality to individual users in an adequate amount, and at a satisfactory pressure. In this section, some basic distribution system design concepts are introduced and discussed.

a. Introduction. Most standard water supply textbooks and many specialized design manuals and handbooks have chapters or sections dealing with the design of distribution systems. Examples are TM 5-813-5, AWWA (1962), Clark, Viessman, and Hammer (1977), Folks (1977), Linsley and Franzini (1979), Merritt (1976), Salvato (1992), Stephenson (1976), Steel and McGhee (1979), and USEPA (1974). The AWWA, Cast Iron Pipe Research Association, National Sanitation Foundation, American Society for Testing Materials, U.S. Army Corps of Engineers, and other professional and technical organizations have developed design techniques, testing and certification procedures, standards, guide specifications, and installation recommendations applicable to pipelines and most distribution system appurtenances. Where applicable, following the recommendations of these organizations will generally result in an adequate design. State and local regulatory agencies usually have rather detailed requirements relative to distribution system design and construction that must be adhered to rigorously. In some cases, distribution system design and construction will be heavily influenced by transportation agencies such as highway departments and railroad companies, and by utilities such as those providing gas, telephone, or electric service. The reason is that water pipelines are usually laid within the rights of way of public highways and roads, and must of necessity cross other rights of way. Traditionally, agencies and utilities such as those mentioned above have insisted on rather conservative pipeline design and construction practices in order to avoid potential interferences with their own activities. Distribution system planners and designers should be aware of this situation and make every effort to cooperate fully with the agencies and companies affected from the very outset of project development. To do otherwise is to invite lengthy delays at every stage of the planning/design/ construction sequence.

b. Purpose. In view of the wealth of information and design guidance already available, the primary purpose of this section is to call attention to some specific points, or factors, that should be considered in the design of small water distribution systems rather than to present detailed design procedures. Since the distribution system often represents the bulk of the capital investment for a water supply system, economic considerations are of a paramount importance.

c. Design flows and pressures. A water distribution system should be capable of delivering the maximum instantaneous design flow at a satisfactory pressure. While exactly what constitutes a satisfactory pressure depends upon system-specific considerations, a typical minimum value is 140 kPa (20 psi). In emergency situations, for example a major fire, system pressures as low as 70 kPa (10 psi) may be acceptable. Absolute maximum allowable pressures are dictated by the pressure ratings of the pipes and appurtenances used and regulatory requirements. However, system pressures should be kept as low as is commensurate with the needs of water users. Unnecessarily high pressures are wasteful in terms of the extra costs of the equipment and energy required to produce them, and the increased volume of water lost to leakage. For most small water systems there is no compelling need for the maximum pressure to exceed 410 or 480 kPa (60 or 70 psi). Thus, a typical approach is to initially design distribution piping for pressures ranging from about 280 to 410 kPa (40 to 60 psi) at the peak hourly flow rate, and then check to see if the design is still adequate at the peak instantaneous flow. A trial and error approach may be used until both conditions are satisfied. Where fire protection is provided, the fire flow will usually govern the design. When absolutely necessary, pressure reducing valves can be used to limit maximum pressures in low-lying areas. However, breaking a small distribution system into multiple pressure zones should be avoided if possible. The estimation of design flow rates is covered in detail in Chapter 4.

d. Pipe sizes. Pipe sizes are ordinarily selected so that flow velocities will range from 0.6 to 1.5 m (2 to 5 ft) per second at design flow rates. However, many regulatory agencies insist on certain minimum pipe diameters and practice oriented rules of thumb for sizing pipes. Where fire protection is provided, it is a good idea to avoid using pipes smaller than 150 mm (6 in.) in diameter. Where no fire protection is provided, pipes as small as 50 mm (2 in.) in diameter may be used. In either case, final pipe size selection should be based upon a complete hydraulic analysis of the system and not solely upon rules of thumb or required minimum diameter.

e. System layout. Textbooks generally call for distribution piping to be laid out in a grid pattern with pipes interconnected at intervals varying from 90 to 360 m (300 to 1200 ft). It is usually also recommended that feeder mains be looped whenever possible. This type of layout is highly desirable because, for any given area on the grid, water can be supplied from more than one direction. This results in substantially lower head losses than would otherwise occur and, with valves located properly, allows for minimum inconvenience when repairs or maintenance activities are required. Unfortunately, grid systems are practical only when water users are distributed more or less uniformly in a grid pattern (e.g., in city blocks). Thus, for small water systems, branching type distribution systems are more common. Nevertheless, it is good practice to loop or interconnect pipes whenever feasible. Normally, underground piping should be located along streets, roads, or utility strips. Minimize locating waterlines under paved areas as much as practical. A typical design approach is to sketch the tentative location of all pipes, connections, hydrants, valves, etc., on a map of the area to be served. Then, using the design flow rates and velocities discussed in AWWA (1962), tentative pipe sizes can be selected. A complete hydraulic analysis can then be performed and pipe sizes and location can be revised until a suitable design is obtained.

f. Hydraulic analysis. The hydraulic analysis of a water distribution system usually involves the use of the Hazen-Williams or Darcy-Weisbach equations to determine frictional head losses in the various pipes and appurtenances for various design flow rates. This information can be combined with topographical data to estimate operating pressures at various locations within the system.

(1) Friction losses. Movement of any fluid through a conduit results in a resistance to flow. This resistance or energy loss is referred to as friction and is usually measured in units of length (meters or feet) or pressure (kPa or psi). As mentioned above, the two most common equations applied to friction loss determination are the Hazen-Williams and Darcy-Weisbach forms. The use of the Darcy-Weisbach equation can provide the best, most reliable solutions for pipe flow problems. However, the roughness of the pipe is still an unknown making the empirical Hazen-Williams equation of equal uncertainty. For direct hand calculations, the determination of friction factor, f is more time-consuming than the direct substitution used in the Hazen-Williams equation. The designer must ensure that proper coefficients and exponents are used depending on whether computations are in the metric or English systems. Virtually all recent engineering textbooks and handbooks provide detail coverage of hydraulics analysis using these two equations.

(2) The Hazen-Williams Equation. A commonly used form of the Hazen-Williams equation is

$$V = 0.85\,(C)\,(R)^{0.63}\,(S)^{0.54} \tag{7-2}$$

where

V = the flow velocity in meters per second

C = a coefficient depending upon the smoothness of the interior of the pipe

R = the hydraulic radius of the pipe in meters

S = the dimensionless slope of the energy grade line

For circular pipes flowing full (as is almost always the case in water distribution systems) the hydraulic radius, in feet, is given by

$$R = D/48 \tag{7-3}$$

where D is the pipe diameter in inches. The dimensionless slope of the energy grade line, S, can be represented as

$$S = h/L \tag{7-4}$$

where

h = the frictional head loss in the pipe

L = the length of the pipe

Obviously, for S to be dimensionless, h and L must be expressed in the same units of length. The Hazen-Williams equation is easily manipulated with the aid of a small calculator; however, virtually all standard water supply engineering textbooks and handbooks provide nomographs that may be used with sufficient accuracy.

(3) Selection of friction factors. The C factor, or coefficient, used in the Hazen-Williams equation reflects the relative smoothness of the inside surface of the pipe under consideration. Typical values range from about 100 for 20-year-old cast iron, to about 130 for asbestos-cement, to 140 or more for plastic pipe. Most water supply textbooks and handbooks provide guidance in selecting C factors (Clark, Viessman, and Hammer 1977; Folks 1977; Lamont 1981; Linsley and Franzini 1979; Merritt 1976; Stephenson 1976; and Steel and McGhee 1979).

(4) Complex systems. Application of the Hazen-Williams equation to single pipelines or small branching- type distribution systems is straightforward and may be readily accomplished by direct hand calculations. However, for more complicated looped and interconnected grid-type systems, some form of network analysis is needed to predict operating pressures. By far the most commonly used technique is that developed by Hardy Cross. The Hardy Cross method is amendable to both small and large systems and is readily computerized. Many textbooks and handbooks (e.g., Clark, Viessman, and Hammer 1977; Linsley and Franzini 1979; Merritt 1976; and Steel and McGhee 1979) present detailed instructions for the use of the method and include worked example problems.

(5) Minor losses. Minor losses are frictional head losses associated with pipe bends, elbows, tees, valves, hydrants, and other distribution system fittings and appurtenances. For long pipelines, these losses are generally negligible. However, in pumping stations, treatment plants, and other locations where equipment is concentrated or piping layouts are complex, they can be substantial. Guidance needed to estimate minor losses is abundant in the water supply literature (e.g., AWWA; Clark, Viessman, and Hammer 1977; Folks 1977; Linsley and Franzini 1979; Merritt 1976; Rao 1982; Stephenson 1976; Steel and McGhee 1979; and Warring 1982).

(6) Water hammer. When the velocity of flow in a pipe changes suddenly, surge pressures are generated as some, or all, of the kinetic energy of the fluid is converted to potential energy and stored temporarily via elastic deformation of the system. As the system "rebounds," and the fluid returns to its original pressure, the stored potential energy is converted to kinetic energy and a surge pressure wave moves through the system. Ultimately, the excess energy associated with the wave is dissipated through frictional losses. This phenomenon, generally known as "water hammer," occurs most commonly when valves are opened or closed suddenly, or when pumps are started or stopped. The excess pressures associated with water hammer can be significant under some circumstances. For example, the maximum pressure surge caused by abruptly stopping the flow in a single pipe is given by

$$a = \frac{4660}{\left(1 + \frac{kd}{Et}\right)^{0.5}} \qquad (7\text{-}5)$$

where

k = bulk modulus of the fluid, pounds per square inch

d = internal diameter of the pipe, inches

E = modulus of elasticity of the pipe materials, pounds per square inch

t = thickness of the pipe wall, inches

As illustrated by Equation 7-5, the magnitude of the maximum potential water hammer pressure surge is a function of fluid velocity and the pipe material. In water distribution systems, water hammer is usually not a problem because flow velocities are typically low (___ to ___ m (3 to 5 ft) per second), and an allowance for surge pressure is built into the pressure ratings of commonly used pipe materials. In the case of hydro-pneumatic systems, there is an extra margin of safety since the pressure tank acts as a buffer against pressure surges. When higher than normal flow velocities are expected, consideration should be given to the use of slow-operating control valves, safety valves, surge tanks, air chambers, and special pump control systems. Since estimation of surge pressures for complex systems involving interconnected pipes and hydraulic equipment can be very involved, it is usually best to obtain the services of an expert in the area of analysis of hydraulic transients when it is anticipated that water hammer may be a problem. Chaudhry (1979) has presented a rather complete discussion of various transient hydraulic phenomena, including water hammer.

g. Pipe materials. The most commonly used water distribution pipe materials are wrought or ductile iron, asbestos-cement, and various plastics. Galvanized steel, copper, and polyethylene are often used for individual water services. The choice of pipe, or service line, material is usually based upon a combination of factors including cost, local availability, bedding conditions, maintenance requirements, ease of installation, and regulatory requirements. With regard to this last point, many agencies and utilities are reluctant to approve the use of plastic pipe and service lines because other agencies and utilities have reported serious problems with them. While it is probable that most of these difficulties have resulted from poor quality control at manufacture; improper storage, handling, and installation; or operational conditions that were not properly considered and accounted for in design, care should be exercised to rigorously follow the recommendations of the AWWA (1980a) when designing polyvinyl chloride (PVC) pipe systems. In critical applications, where replacement would be especially difficult or expensive, it would seem prudent to avoid the use of plastic pipe. Regardless of the choice, the pipe, or service line, should confirm to the applicable AWWA and National Sanitation Foundation Standards, as well as local regulatory requirements.

(1) Distribution pipes. Historically, cast iron has been the most popular type of pipe for water distribution system applications. However, in recent years the plastics, especially PVC, have become increasingly popular for small distribution

systems. Advantages of PVC include the typically lower cost, light weight, ease of installation, and virtual immunity from corrosion. Some regulatory agencies and water utilities have not, however, approved the use of plastic pipe within their jurisdictions. Discussions of the advantages and disadvantages of various pipe materials are presented in many textbooks, handbooks, and design manuals (Folks 1977; Merritt 1976; Stephenson 1976; Steel and McGhee 1979; and Warring 1982). AWWA (1980a) and the Cast Iron Pipe Research Association (1978) have presented excellent discussions of the design and installation of PVC and cast and ductile iron pipe, respectively.

(2) Service lines. Traditionally, copper and galvanized steel have been used for water services. However, recently plastic tubing, especially polyethylene (PE), has become popular for small water systems. The major advantages of PE tubing are its relatively low cost, corrosion immunity, and ease of installation. This last point is especially important because PE tubing can be installed without the use of the special gooseneck connectors needed for more rigid materials.

h. Valves. Several types of valves may be used in water distribution systems. Four common types are discussed below. The locations of all valves should be clearly marked on as-built plans, and described in relation to readily identifiable landmarks or prominent physical features, so that they can be easily found in the field. All valves should be protected by suitable valve boxes (usually cast iron, concrete, or high-density plastic) and located so that they will not be affected by normal street or highway maintenance operations. Warring (1982) has presented an excellent discussion of various types of valves.

(1) Isolating valves. Valves are needed to allow portions of the distribution system, fire hydrants, storage tanks, and major hydraulic equipment to be isolated for repairs and maintenance with minimal disruption of system operation. Double disk gate valves are usually used for this purpose since they are widely available, relatively low in cost, create very little head loss in the fully open position, seat dependably, and effectively stop flow in the fully closed position. They are, however, of only limited value for throttling or controlling flow and are, therefore, not usually used for such purposes. Butterfly valves are commonly used when throttling or flow control is desired. Gate valves should be placed at all pipe intersections and on all pipe branches. On long pipe runs, gate valves should be spaced no more than ___ km (1 mile) apart.

(2) Air relief and vacuum valves. Air tends to accumulate at high points along waterlines and can significantly interfere with flow, especially on longer lines. Therefore, air relief valves should be placed at all high points on long waterlines. Manually controlled valves are available, but it is much more common to use the automatic type. The exact locations of air relief valves should be determined in the field as the pipe is being installed. Valves are also needed to protect pipelines from collapse as they are emptied, by allowing air to enter the pipes. Vacuum valves are used for this purpose. Combination air relief-vacuum valves are available. Air relief and vacuum valves are normally not needed within interconnected grid portions of distribution systems.

(3) Flushing valves. Flushing valves, or hydrants, are needed at the ends of all dead-end lines. They serve a dual purpose: to release air as lines are filled, and to allow occasional flushing to remove sediment that invariably accumulates at dead ends. The simplest type consists of an ordinary gate valve to which a short piece of pipe can be attached when needed. The function of this length of pipe is merely to direct the flow as desired to avoid excessive erosion and other related problems. Since flushing and/or filling lines are needed only occasionally, manually operated valves are sufficient.

(4) Pressure reducing valves. Occasionally, topography will be such that excessive pressures result in low-lying regions of the distribution system. In such cases, pressure reducing valves can be quite useful. They operate automatically to throttle flow to maintain the desired downstream pressure as long as the upstream pressure is sufficient. For small systems, it is generally best to avoid using pressure-reducing valves on distribution lines if at all possible. Pressure reducing valves are frequently used on individual water service lines to protect house plumbing and appliances such as water heaters.

i. Fire hydrants. When fire protection is provided, hydrants meeting the requirements of the AWWA should be installed. Generally, fire hydrants should not be located on mains smaller than ___ mm (6 in.) in diameter, and should be connected to the main by a short run of ___-mm- (6-in.-) diameter pipe controlled by a gate valve. In operation, this valve should always remain open unless it is necessary to prevent flow to the hydrant. Hydrants should never be installed on lines that are unable to supply an adequate flow. When the hydrant will be exposed to possible damage from vehicular impact, the type that is designed to fail near the ground level and minimize the chance of damage to the distribution system (and the resulting water loss) is preferred.

j. Water meters. Several types of water meters are available. The rotor type has become increasingly popular since models that are accurate at very low flow rates were introduced. Complete details for meter selection and installation are presented by the AWWA (1962).

k. Thrust blocking. Thrust blocks are used to prevent the movement of pipes and appurtenances that would otherwise result from changes in flow rate or direction, or unbalanced pressure forces. They are needed at changes in alignment (e.g., tees, bends, elbows, and crosses), wherever reducers are used, at stops or dead ends, and at valves or hydrants where thrust develops when flow is started or stopped. Poured-in-place concrete is usually used. Many design methods and nomographs are available to help designers size blocks for specific situations. Standard techniques are presented in Appendix A. Thrust blocks are usually designed using methods similar to those used to design foundations and footings. Factors that affect design include pipe or appurtenance size, maximum operating pressure, type of fitting or appurtenance, pipeline profile, and soil bearing capacity.

l. Loads on pipes. Loads that may be superimposed on buried pipes generally fall into two categories: earth loads and live loads. When calculating the total load on a pipe, separate earth and live load analyses are usually performed and the results summed. The commonly used methods of estimating earth loads are based upon theories originally proposed by Anson Marston in the early 1900s. Suitable techniques are presented in references in Appendix A. Live loads generally result from vehicular traffic and are often insignificant when compared to earth loads. Exceptions arise when pipes are placed at shallow depths underneath roadways. The exact meaning of the term "shallow" is controlled by site-specific conditions. However, as a rule, live loads diminish rapidly for laying depths greater that about ___ m (4 ft) for highways and ___ m (10 ft) for railroads. Information needed to estimate live loads resulting from various standard loading criteria are presented in references in Appendix A. In many cases, local regulatory or transportation agencies and utilities will have rather restrictive rules concerning waterlines crossing roads and railroads.

m. Boring and casing. It is common for regulatory and transportation agencies and utilities to require that steel pipe casings be used when waterlines cross highways and railroads or other rights-of-way. Casings protect roadbeds from excessive damage during construction, or when failures occur and repairs must be made; limit the inconvenience associated with construction, failures, and repairs; and may be cheaper than excavating and backfilling, especially when expensive roadway surfaces must be replaced. Casings are usually installed by boring and jacking. The agency or utility that requires boring and casing will generally have specific requirements governing the type and size casing to use. However, casings generally must be ___ to ___ mm (4 to 8 in.) larger in diameter than the waterline to accommodate the pipe joints. Pipes placed in casings should be supported so that the weight of the pipe (and water) is not borne solely by the joints.

n. Pipe laying. Specific instructions for laying various types of pipe are presented in references in Appendix A. It is important for the designer to make sure that the contract documents are written to clearly specify installation procedures. Important points that should be addressed are pipeline alignment (vertical and horizontal) and trench construction. Generally, trenches should be kept as narrow as is commensurate with installation of the given pipe size, adequate clearance should be given to sewer lines, and some minimum pipe cover should be maintained. Minimum cover of ___ to ___ m (3 to 4 ft) is commonly specified, except in very cold areas. All pipes should be bedded so that uniform longitudinal support is provided. Care should be taken to see that the pipe and connections are not damaged during laying, all appurtenances are properly installed, adequate thrust blocking is provided, and the trench is properly backfilled. Regulatory agencies may have their own requirements, but generally rely on manufacturers' recommendations and the appropriate AWWA Standards.

o. Disinfection. Distribution systems should be disinfected prior to being placed in service. The commonly used methods are presented in the AWWA Standards. Contract documents should require that care be taken, during both the storage and construction periods, to prevent excessive contamination from occurring, and should specify the disinfection procedures to be used. In general, disinfection involves flushing with clean water, heavy chlorination for an extended period (usually 24 hours), flushing again with clean water, and bacteriological testing to confirm the efficacy of the process.

p. Testing. New waterlines must be tested to ensure that they will hold the specified pressure and not leak more than some specified amount. Usually the tests are conducted simultaneously. Step-by-step procedures are given in the AWWA Standards for the particular type of pipe being tested. Testing should be performed after thrust blocks have developed adequate strength (usually 7 days), but before the trench is backfilled, except that some backfilling may be needed to hold the pipe in place and prevent incidental damage. In general, the procedure calls for the pipe to be gradually filled with clean water; for all air to be expelled; for the test section to be isolated by capping, plugging, or closing valves; and for the section to be connected to a pump capable of maintaining the desired pressure (plus or minus ___ MPa (5 psi). The typical test duration is 2 hours. During this period, the pipe and all appurtenances should be carefully inspected. Any visible leaks should be repaired and the section retested. The pressure should not drop by more than ___ MPa (5 psi) and the leakage should be less than or equal to the volume calculated as follows:

$$L = \frac{(N)(D)(P)^{0.5}}{7400} \tag{7-6}$$

L = the maximum allowable leakage in gallons per hour

N = the number of joints in the length of the pipe being tested

D = the nominal diameter for the pipe in inches

P = the average test pressure (gauge pressure) in pounds per square inch.

The test pressure is usually the greater of 150 percent of the working pressure at the test location or 125 percent of the working pressure at the highest elevation along the section of line being tested.

Appendix A
References

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TM 5-813-3
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TM 5-813-4
Water Supply, Water Storage

TM 5-813-5
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ER 200-2-3
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